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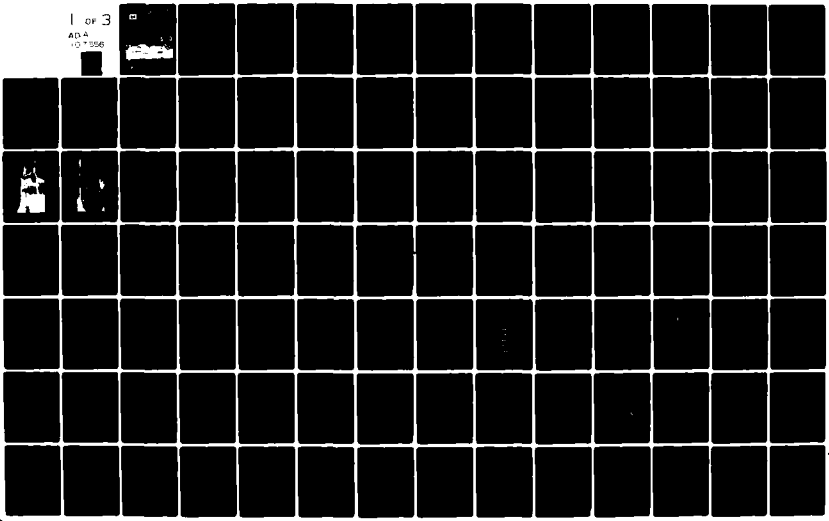

TECHNICAL REPORT EL-81- 7

# DESIGN, CONSTRUCTION, AND ANALYSIS OF FABRIC-REINFORCED EMBANKMENT TEST SECTION AT PINTO PASS, MOBILE, ALABAMA

by

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20. ABSTRACT (Continued).

bearing of approximately 0.4 to 0.6 was found to exist for conventional construction. The embankment would have to be raised an additional 17 ft during the 10 years after construction, and an additional 25 ft in the next 20 years. Fine, poorly graded sand was available nearby for use as borrow material in initial phases of dike construction. Nonengineering considerations dictated that the embankment be constructed with all deliberate speed and with minimal possibility of failure.

After considering alternatives of preloading, use of lightweight construction material, and end-dumping displacement, it was decided to attempt construction of a civil engineering fabric-reinforced embankment along the alignment. The civil engineering fabric would be placed between the soft cohesive foundation and the sand embankment material in long, narrow strips perpendicular to the alignment, with the strips overlapped and sewn together. The fabric was postulated to act as tensile reinforcement, maintaining the embankment in a coherent mass, and partially supporting embankment weight until consolidation of soft foundation materials could occur.

The concept, while simplistic, nevertheless required proper selection of geotechnical fabric and appropriate embankment design and construction procedures to obtain satisfactory performance. It was decided to evaluate the concept by construction of an 800-ft-long embankment test section. Criteria used in preliminary design of the test section are presented herein, as are summary results of laboratory testing conducted to select a proper reinforcement fabric. Construction of the test section is documented and actual behavior is compared with behavior predicted prior to construction. Also, revised design criteria to be used for the remaining 5200 ft of fabric-reinforced embankment and applicable to other conditions are described.

It was concluded that the use of fabric as an embankment reinforcement is a viable and innovative construction technique that will enable geotechnical design and construction of embankments on extremely soft foundations.

## PREFACE

This report describes the design, construction, and analysis of a full-scale embankment test section at Pinto Pass, Mobile, Alabama. This project was conducted as a cooperative effort of the U. S. Army Engineer District, Mobile (MOD), and the Dredging Operations Technical Support (DOTS) Program, the U. S. Army Engineer Waterways Experiment Station (WES), Vicksburg, Mississippi. The DOTS is sponsored by the Office, Chief of Engineers, through the Dredging Division of the Water Resources Support Center, and is managed by the WES Environmental Laboratory (EL).

Research described in this report is an extension of the work relative to the design of confined disposal operations proposed during the Dredged Material Research Program (DMRP). Previous reports prepared during the DMRP provided the guidelines for the feasibility for the design and construction of a fabric-reinforced embankment constructed on soft foundation materials. This report provides the data on design and construction methodology for cost-effective dredged material containment embankments.

Concept formulation, general supervision of the research, and construction of the fabric-reinforced embankment were conducted by Mr. Jack Fowler, WES Geotechnical Laboratory (GL), with the assistance of Dr. T. Allan Haliburton, DMRP Geotechnical Engineering Consultant. This report was presented to Oklahoma State University as Mr. Fowler's Dissertation for a Doctor of Philosophy in Civil Engineering.

Onsite research operations were assisted by Mr. Ken Jackson, MDO, Mobile Area Office. Mr. J. Patrick Langan, Assistant Chief, MDO Project Operations Branch, was responsible for contractual details along with general assessment and guidance in the conduct of the work.

This report was written by Mr. Fowler under the general supervision of Mr. Charles C. Calhoun, Jr., DOTS Program Manager, and Dr. John Harrison, Chief, EL. This report was also prepared under the general supervision of Mr. G. B. Mitchell, Chief Engineering Group, Soil Mechanics Division (SMD); Mr. C. L. McAnear, Chief, SMD; and Mr. J. P. Sale, Chief, GL.

District Engineer of the MDO during this period was COL Charlie L. Blalock, CE. Commanders and Directors of the WES were COL John L. Cannon, CE, and COL Nelson P. Conover, CE. Technical Director of the WES was Mr. F. R. Brown.

This report should be cited as follows:

> Fowler, J. 1981. "Design, Construction, and Analysis of Fabric-Reinforced Embankment Test Section at Pinto Pass, Mobile, Alabama," Technical Report EL-81-7, U. S. Army Engineer Waterways Experiment Station, CE, Vicksburg, Miss.

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CONVERSION FACTORS, U. S. CUSTOMARY TO METRIC (SI)
UNITS OF MEASUREMENT

U. S. customary units of measurement used in this report can be converted to metric (SI) units as follows:

| Multiply | By | To Obtain |
|---|---|---|
| cubic yards | 0.7645549 | cubic metres |
| Fahrenheit degrees | 5/9 | Celsius degrees or Kelvins* |
| feet | 0.3048 | metres |
| gallons (U. S. liquid) | 3.785412 | cubic decimetres |
| inches | 25.4 | millimetres |
| kips (force) per square foot | 47.88026 | kilopascals |
| pounds (force) per foot | 14.5939 | newtons per metre |
| pounds (force) per inch | 175.1268 | newtons per metre |
| pounds (force) per square foot | 47.88026 | pascals |
| pounds (force) per square inch | 6894.757 | pascals |
| pounds (mass) | 0.4535924 | kilograms |
| pounds (mass) per cubic foot | 16.01846 | kilograms per cubic metre |
| square yards | 0.8361274 | square metres |
| tons (force) per square foot | 95.76052 | kilopascals |

* To obtain Celsius (C) readings from Fahrenheit (F) readings, use the following formula: $C = (5/9)(F - 32)$. To obtain Kelvin (K) readings, use $K = (5/9)(F - 32) + 273.15$.

# CHAPTER I

# INTRODUCTION

## Background

Traditionally, the Corps of Engineers (CE), has been responsible for developing and maintaining the Nation's navigable waterways and harbors and, in this capacity, has been responsible for disposal of large quantities of dredged material. Heretofore, the CE had been able to deposit the material removed by dredging activities into open-water and land-based sites, but recent growth of national environmental concern has resulted in the restriction of open-water and other unconfined disposal of dredged material. If all dredging were terminated, especially maintenance dredging, disastrous effects on the Nation's commerce and economy would result. To remedy this potentially conflicting situation, the Dredged Material Research Program (DMRP) of the CE was initiated to determine the environmental effects of the dredging and disposal process and to develop environmentally compatible, technically feasible, and cost-effective methods for dredging and dredged material disposal, including reclamation and use of dredged material as a resource.

As part of the DMRP and this study, design and contruction of dredged material containment dikes reinforced with geotechnical fabric and constructed on soft foundation materials were investigated. (Geotechnical fabric is a generic term applied to a wide variety of artificial fiber

textile products used in engineered construction of civil works; also called civil engineering fabric, geo fabric, geo testiles, and filter cloth.)[1] The U. S. Army Engineer District, Mobile, Alabama (MDO), initiated a feasibility study in a cooperative effort with the U. S. Army Engineer Waterways Experiment Station (WES) to investigate the applicability of this type of design and construction technology.

Haliburton, Douglas, and Fowler concluded that the use of Pinto Island as a long-term disposal site would be contingent on construction of a 6000-ft-long* dredged material containment dike across and along Pinto Pass.[2] Foundation along the dike alignment consisted of soft clays with liquid limits of about 100 and undrained shear strengths ranging from 50 to 150 psf. After a consideration of several alternative methods, it was concluded that a fabric-reinforced multipurpose dike would be constructed for the Pinto Pass disposal area. The dike would act as an initial containment structure to el 8, as a preload structure to facilitate rapid incremental construction to el 25, and as a substructure for long-term raising to el 50. (Note: All elevations are given in feet referenced to National Geodetic Survey datum.)

Since only minimal data existed in manufacturers' literature for designing fabric-reinforced embankments, the MDO contracted with the School of Civil Engineering at Oklahoma State University to conduct testing and to develop data required for proper evaluation and selection of fabrics for use in dike reinforcement (dike and embankment are used interchangeably in this report). Haliburton, Anglin, and Lawmaster tested and evaluated 27 commercially available petrochemical-based geotechnical fabrics and one fiberglass fabric for possible use in an

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* A table of factors for converting U. S. customary units of measurement to metric (SI) units is presented on page xiii.

embankment test section to be constructed across Pinto Pass.[3] Four woven fabrics were selected and subsequently used in construction of an 800-ft-long trial test section across the west end of Pinto Pass.

## Purpose

The purpose of this study was to develop and evaluate criteria for design, construction, and analysis of a fabric-reinforced earth embankment test section constructed on soft foundation materials.

## Scope

The scope of the study included collection and evaluation of data from a fabric-reinforced sand embankment test section constructed on a clay (CH) foundation, determination of the technical feasibility of the concept, and verification of preliminary fabric-reinforced embankment design criteria for use in future projects.

ENDNOTES

[1] T. Allan Haliburton, Cyd C. Anglin, and Jack D. Lawmaster, "Selection of Geotechnical Fabrics for Embankment Reinforcement" (School of Engineering, Oklahoma State University, Stillwater, Oklahoma, 1978), p. 2. Prepared under Contract DACW01-78-C-0055 for the U. S. Army Engineer District, Mobile.

[2] T. Allan Haliburton, Patrick A. Douglas, and Jack Fowler, "Feasibility of Pinto Island as a Long-Term Dredged Material Disposal Site," Miscellaneous Paper D-77-3 (U.S. Army Engineer Waterways Experiment Station, Vicksburg, Mississippi, 1977), p. 27-28.

[3] Haliburton, Anglin, and Lawmaster, p. 11.

## ENDNOTES

[1]T. Allan Haliburton, Cyd C. Anglin, and Jack D. Lawmaster, "Selection of Geotechnical Fabrics for Embankment Reinforcement" (School of Engineering, Oklahoma State University, Stillwater, Oklahoma, 1978), p. 2. Prepared under Contract DACW01-78-C-0055 for the U. S. Army Engineer District, Mobile.

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[3]Haliburton, Anglin, and Lawmaster, p. 11.

## CHAPTER II

## STATEMENT OF PROBLEM

Construction of about 6000 lin ft of 8-ft-high embankment on extremely soft foundation materials would be necessary as part of the proposed Pinto Pass dredgeu material containment area. About 50 percent of the alignment was in the intertidal zone, on soft cohesive foundation materials with undrained shear strengths ranging from 50 to 150 psf.

After careful consideration of various construction alternatives of preloading, use of lightweight construction materials, and end-dumping displacement, it was decided to attempt construction of a floating fabric-reinforced embankment. A geotechnical fabric would be placed between the cohesive foundation and an embankment constructed with poorly graded sand from dredged material disposal areas located nearby. The fabric was to be laid perpendicular to the alignment of the dike in long, narrow strips and sewn at each overlap. It was postulated that the fabric would perform like tensile reinforcement in a long soft concrete beam. maintaining the embankment in a coherent mass and supporting the embankment until consolidation of the underlying soft cohesive soils had occurred. To verify the concept, an 800-ft-long test embankment would be built, and test section results would be used to design the remaining prototype embankment.

The general location of the proposed embankment test section relative to the City of Mobile and Mobile Harbor, Alabama, is shown in

Figure 1. Figure 2 is an aerial view of Pinto Pass, while Figure 3 is a preconstruction ground view of the area where the test section was built. It may be noted in the photographs that the proposed test section was constructed across an intertidal area; existing ground elevations over most of the alignment ranged between el 1.5 and el -1.0.

It was decided to evaluate an 800-ft-long test section reinforced with four different woven geotechnical fabrics selected from the 28 fabrics tested by Haliburton, Anglin, and Lawmaster.[1] Criteria for design of the test section, summary of the laboratory fabric tests conducted, construction techniques, and behavior of the embankment predicted before construction were to be evaluated. In the process of designing the test section, considerable research was conducted, fabrics evaluated, and preliminary design and construction technology developed. Since the potential widespread applicability of this construction technique in the CE for raising dikes on soft foundations could significantly expand civil engineering design concepts in this field, methods of proper design and construction of fabric-reinforced embankments on soft foundations were to be developed and verified for use by all CE elements.

ENDNOTE

[1]T. Allan Haliburton, Cyd C. Anglin, and Jack D. Lawmaster, "Selection of Geotechnical Fabrics for Embankment Reinforcement" (School of Engineering, Oklahoma State University, Stillwater, Oklahoma, 1978), p. 11. Prepared under Contract DACW01-78-C-0055 for the U. S. Army Engineer District, Mobile.

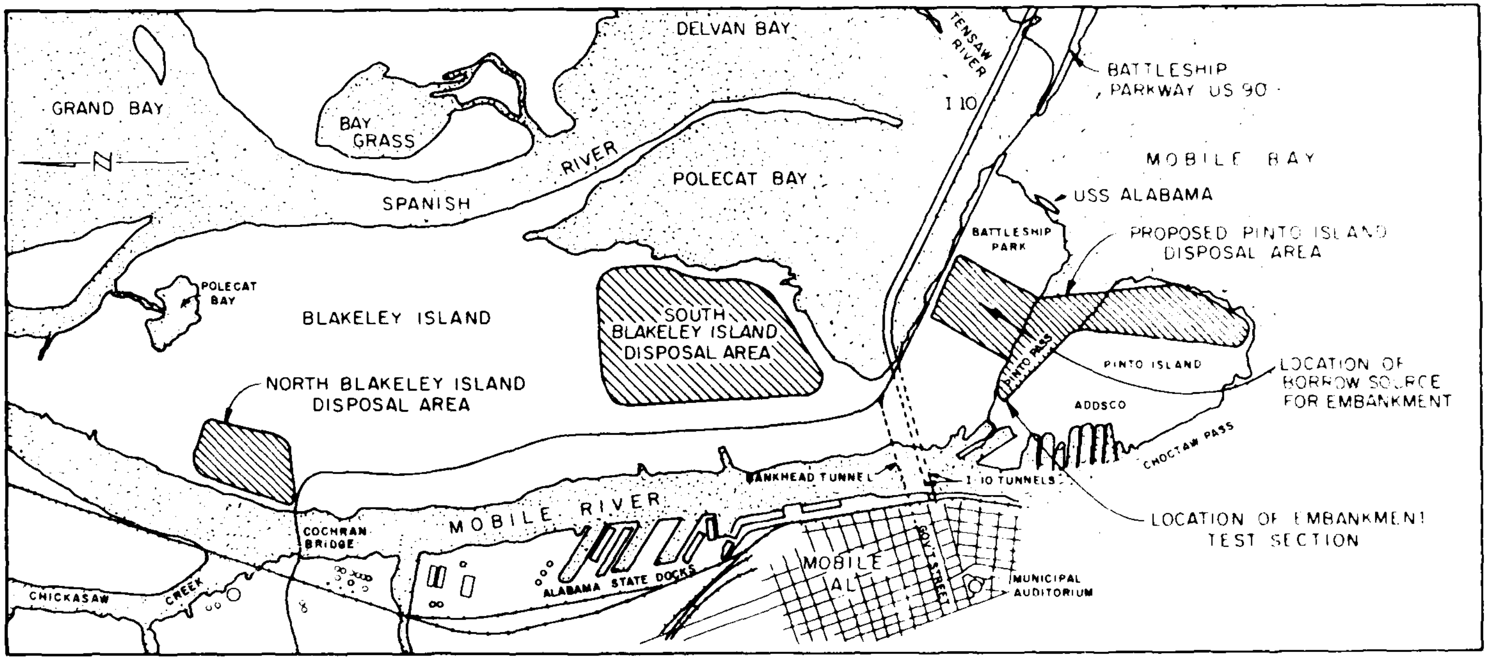



Figure 1. General Location of Embankment Test Section in Mobile Harbor, Alabama

Figure 2. Aerial View of Pinto Pass Looking East. Embankment Test Section Was Constructed At Pinto Pass at About Top Quarter Point of the Photograph

Figure 3. Ground Photograph of Pinto Pass with Mobile Harbor in the Background. Embankment Test Section to Be Constructed East of the Narrow Channel Shown in the Photograph

# CHAPTER III

## HISTORICAL AND LITERATURE REVIEW

### Introduction

Historically, a great many materials have been used in attempts to reinforce or separate embankments or roadways from soft underlying foundation materials. In modern times, the more commonly occurring raw materials and manufactured products have been used, but, over the last two decades, synthetic fabrics have been found to be more economical, more easily handled, stronger, and longer lasting than many traditionally used materials. Further, these synthetic fabrics resist a large range of acid and basic soils and liquids (natural and man-made) as well as biological attacks.

In the near and distant past, in both hemispheres, reinforcement or separation has been achieved using animal skins, bamboo, cane, rushes, grasses (straw), willow branches, logs, poles, boards, steel and aluminum mats, wool, cotton, and jute, in various manners of placement. These materials were woven, nailed, welded, bolted, tied in mats, sewn together, and laid individually on the surface or embedded in the soil to provide a more stable embankment (see Table I).

Unfortunately, many of the natural staples do not afford the protection that the synthetic fabrics provide. To control seepage, cotton was used in the lining of reservoirs built earlier in this century, but rapid breakdown caused by sunlight and organisms in the soil made

TABLE I

FIBROUS MATERIALS USED AS REINFORCEMENT

| Type | | Composition | |
|---|---|---|---|
| Natural | Animal | | Wool, Silk (Protein) |
| | Vegetable | | Cotton, Jute (Cellulosic) |
| Regenerated | Organic | | Rayon, (Carbon) |
| | Inorganic | | Glass, (Steel) and Aluminum |
| Synthetic | Organic | Polyamide | Nylon |
| | | Polyester | Terylene |
| | | Polyolefin | Polypropylene<br>Polyethylene |
| | | Polyvinyl | Acrylic<br>PVC |

further use of this technique impractical; and attempts to increase the longevity of the cotton by treating it with such chemical fungicides as copper or mercury proved to be not only uneconomical, but also detrimental to the environment.

Jute has also been used extensively, but it is subject to rapid decay as a result of its water-absorptive nature. It has been used successfully and economically, however, in projects requiring high strength. The strength is provided by intertwining fine steel strands with the jute thread prior to weaving.

## Permeable Synthetic Fabrics

Woven permeable synthetic fabric membranes have been used in Europe and the United States for the past 25 years in civil engineering applications associated with soil. Nonwoven permeable synthetic fabrics have been used in these same applications for about a decade. Considerable research has been conducted throughout the world since the geotechnical applications of synthetic fabric were realized. Research to determine the behavior and mechanics of soil-fabric systems, so that better specification and product development can be achieved, has been conducted by both producers and users of these fabrics.

In 1973, McGown and Ozelton determined that, for soil engineering applications, permeable fabrics have three basic operational functions: (1) separation, (2) filtration, and (3) reinforcement.[1] In 1974, Leflaive and Puig added a fourth function: drainage in the plane of the fabric.[2,3] Although the other factors are relevant, reinforcement and separation are considered to be of prime importance in determining the strength of a soil-fabric mass.

Separation of two or more different types of materials is achieved with the fabric to prevent intermixing and consequent change in the geotechnical behavior of each material. Reinforcing the soil with fabric provides the soil with a tensile load-carrying capacity that causes a change in the stress-strain patterns within the soil. This is in contrast to normal design considerations in which soil is assumed to have very low tensile strength.

## Synthetic Fiber Polymers

Synthetic fibers have created an entirely new dimension in the use of engineering fabric from the standpoint of both pore-size uniformity and range of pore sizes possible. The use of synthetic fibers allows improved production control of fabrics, with greater rot and mildew resistance, flexibility, tensile strength, chemical resistances, and lower water-absorbtive characteristics than those produced from natural fibers (see Table I). The types of synthetic fibers used to construct geotechnical fabrics are primarily polymers that are constructed in many different blends and combinations, which have individual advantages and disadvantages for various applications.

Synthetic fiber polymers are products of the petroleum industry and are derived from propylene and ethylene gases. The polyolefins are the primary constituents of polypropylene and polyethylene. Polyethylene was produced in 1941 in its original form and primarily manufactured in monofilaments for webbing and cord-type applications. Polypropylene was developed in fiber form in 1954 and exhibited better physical properties at lower densities than polyethylene. Polypropylene has a specific gravity in the range of 0.90 to 0.92, the lowest for any plastic material.

Polypropylene has a good balance of physical and chemical properties and is as economical to produce as most other plastic materials. It is second only to Teflon in its resistance to alkalis, acids, and oxidizing and reducing agents. Like polyethylene, it is also resistant to organic solvents such as ethyl acetate, chloroform, and carbon disulfide.

Like polyethylene, polypropylene is subject to ultraviolet (UV) degradation (sunlight), but this tendency can be retarded by the addition of UV absorbers such as carbon black. Creep under load over a period of time, which is a direct function of temperature, has been retarded by a new type of cross-linked polyolefin, and there are still other types not yet available that may have even less creep potential.

Polyesters, developed in the United Kingdom soon after World War II as Terylene, are derived from dihydric alcohol and terephthalic acid. Polyesters are available in a number of fiber forms and shapes, have good to excellent resistance to mineral acids, and are normally insoluble in most common solvents, but are deteriorated by detones and hydrocarbons. The specific gravity of polyesters ranges from 1.31 to 1.38, and most polyesters have good resistance to creep in woven or single strand form. Some of the newer polyesters, which are easily extrudable, have high impact and abrasion resistance.

Polyamide or nylon was developed in 1931 in a search for a superpolymer. The two main types of nylons used for geotechnical fabrics are Nylon 6 and Nylon 6/6, even though there are many other variations in existence or under development. There are some very good nylon products that are comparable to polyolefins and polyesters in tensile strength and wear resistance. Nylons have good tensile strength, flexibility,

and compressive strength characteristics over a temperature range of 32° to 300°F. Specific gravity of Nylon 6 and 6/6 is about 1.14. One of the problems with nylon is that it is very water absorbent and, when employed in a geotechnical project, it will readily absorb any water present, possibly causing placement problems with large fabric sheets. Nylons are also subject to UV deterioration unless protected by UV absorbers such as carbon black or resistant print covering. Nylon 6 and 6/6 will decompose in strong mineral (sulphuric) acid, but like most nylons, are not affected by weak acids or alkalis.

Vinyls are structurally based on the ethylene chain and consist of seven major types: (1) polyvinyl alcohol, (2) polyvinyl acetate, (3) polyvinyl acetol, (4) carbazole, (5) polyvinyl chloride, (6) polyvinylchlorideacetate, and (7) polyvinylidene chloride. The most common and widely used is polyvinyl chloride (PVC), available in resin, latex, organosol, etc., forms.

PVC can be fabricated in sheets, thick to thin, in the form of pipes and pipe fittings, and in wide ranges of rigidity and flexibility. Vinyls are basically strong, tough, and resistant to water and abrasion.

Glass fibers are formed by continuous drawing of glass from a special melt furnace. This process was developed by Owens Corning, Inc., before World War I. Glass fiber was used as a thermal insulator in Germany about the same time and has proven to be resistant to heat, moisture, most acids and alkalis, and most common solvents.

Fiberglass has the ability to overcome the problems of creep and dimensional stability that occur in thermoplastic materials (plastics capable of being repeatedly softened by increases in temperature and hardened by decreases in temperature rather than chemical changes).

Thermoplastics reinforced with fiberglass have been noted to have a modulus of elasticity two-and-one-half times greater than the non-reinforced thermoplastics. Fiberglass has good resistance to heat, softening at 1350° to 1560°F.

Glass fabric may be woven or nonwoven, but the variations of polymer types, the rate of development of fibrous materials, and the methods of forming bicomponents are so rapid that it is difficult to classify and describe them with any accuracy. However, the initial properties exhibited by glass fabrics for geotechnical application are very promising.

## Fiber Physical Forms

Since the technology of fabrics has been developed by the textile industry, the physical forms of the fibers have fallen into three broad categories: staple, tow, and continuous filament. The terms staple and tow were derived in textile industries to describe yarns produced from natural fibers. Staple refers to discontinuous natural fibers or discontinuous synthetic fibers formed by an extruding and cutting process, and is the raw material of the yarn formed in the spinning process for both natural and synthetic fibers.

Tow also originated in the natural fiber processing, but has a different meaning for synthetic fibers. In the manufacture of synthetic fibers, tow is a ropelike strand of continuously extruded fibers that are generally parallel and are without twist. Tow is used particularly for staple in synthetic yarns that resemble conventionally spun forms.

Continuous filament yarn material may be produced by nature or the synthetic fiber process. In nature, the fiber is produced by multiple

spinnerets in the head of a silkworm; in the man-made process, there may be only one orifice in the spinneret, producing a monofilament yarn, or many orifices, creating multifilament yarn by a continuous extrusion process. The spinneret design controls not only the number of filaments in the fiber but also the cross-sectional shape, which can be widely varied. The shape, amount, and manner of twist can also have considerable influence on the filament yarn performance and texture.

One other fiber form in use is a narrow ribbonlike continuous fiber (tape) that is being included increasingly in the construction of fabrics. It is produced in a "monofilament" form by slitting entire roll-widths of extruded film.

## Woven and Nonwoven Fabric Construction

Fabrics are divided into two main classes of woven and nonwoven forms. Woven fabrics are produced by the traditional weaving or knitting process that has been used in the textile industry since the modern industrial revolution (see Table II). Nonwoven fabrics involve various bonding methods such as needle or needlefelt punching, chemical fusion at fiber contact points by various bonding materials, and heat fusion at fiber contact points.

Although natural and synthetic fibers may be used in both woven and nonwoven fabrics, nonwovens are generally formed from synthetics (primarily polyesters, nylons, polypropylenes, and fiberglasses) utilizing one or more of the following techniques:

1. Needle or needlefelt punching is a technique developed for synthetic fibers. A web of fibrous filaments is subjected to the insertion of a number of reciprocating barbed needles that engage and

TABLE II

TEXTILE FABRICS

| Types | | | Characteristic Properties |
|---|---|---|---|
| Woven | | | Flexible, high modulus, good strength |
| Intermediate Woven | Knitted | — warp<br>weft | Flexible, low modulus, extensible |
| Nonwoven | Felted | — natural<br>— mechanical (needle-punched) | average properties<br>— medium to low modulus<br>— medium to high extension |
| | Bonded | — resin<br>— melt<br>— stitch | poor abrasion |
| | Others | — e.g., tufted<br>extruded net | |

mechanically entangle the fibers in a random fashion. The entangled weblike mass may then be shrunk by applying wet or dry heat to give the fabric a dense, feltlike structure.

2. Chemical bonding of the webbed fibrous filaments is accomplished by introducing chemical adhesives such as latices and resins. These chemicals are applied in a liquid form to impregnate and hold the fibers together.

3. Self-bonded, spun-bonded, heat-fused, and welded are all synonymous terms applied to fabrics that are produced by utilizing the thermoplastic characteristics of the fibers. The fibers are brought together in various arrangements, pressed together, and heated to the melting point.

4. A wide variety of composite nonwoven fabrics may be created by combining two or more of the bonding techniques with any number of combinations of fiber types.

Although polyethylene is not used in the nonwoven fabrics, it is used as a component of woven geotechnical fabrics, along with the previously mentioned polyesters, polypropylenes, nylons, and fiberglass.

Woven fabrics are usually constructed from filaments or tapes crossed over at right angles in two or more planes. Woven fabrics are constructed with either mono-, multiple, or plaited filaments. The distances between the links of the woven fabric may vary from 1/50 of a millimetre to as much as a centimetre, and the filaments may be heated together or glazed in some manner. The heavy woven fabrics, composed of bands (polypropylene, tapes, or ribbon) or large-diameter filaments or rope, exhibit a slightly waffled surface. The fabric has a marked thickness, a slightly uneven texture, and a relatively high permeability.

The synthetics such as polyester, polyamide (nylon), and polypropylene fibers used in nonwoven fabrics are also used in constructing woven fabrics.

The ordering and alignment of the fibers and yarns in a woven fabric dictate that the tensile load is shared by the stressed fibers more or less equally. Therefore, a higher percentage of the inherent fiber strength is obtained, with fabric properties reflecting the fiber properties. Consequently, the deformation modulus of woven fabrics is normally high and extensibility is comparatively low.

Lack of this ordered fiber alignment in nonwovens leads to individual fibers being stressed at different levels. Therefore, considerably lower percentages of potential strength and deformation modulus are achieved, but this is usually offset by increased extensibility of the nonwovens before failure or rupture. Knitted fabrics were designed to extend and drape over rough surfaces and consequently have low tensile modulus. There are fabric constructions that fall somewhere between knitted and woven fabrics that have exnibited special benefits, such as high initial modulus because there is no straightening of the fibers to occur during load as with some woven fabrics.

There have also been techniques devised to optimize the tensile characteristics of synthetic materials, by orienting the polymer molecules in such a way that they are aligned in a drawing or extending mode. Consequently, a fabric woven from such micro-engineered yarns will exhibit much greater tensile strength than fabrics of otherwise identical fiber and weave.

## Fabric Selection Criteria

When choosing a fabric type, the cost and mechanical and physical properties that will have the most effect on the civil engineering application must be considered. Cost may be as much of a determining factor as mechanical and physical properties, and cost-effective analysis must also be considered. Mechanical properties to be considered include the shape and magnitude of the stress-strain curve and fabric resistance to effects such as tearing, creep, dynamic loading, fatigue, abrasion, and degradation under environmental conditions. Physical properties of the fabric to be considered are thickness, weight, porosity, and pore-size distribution and also variation of the physical properties during the life of the fabric. These are some of the properties that must be considered before specifications for a fabric can be written, to ensure adequate performance of a properly designed structure.

Determining the required specification for a geotechnical fabric is not an easy task. There are currently no generally accepted standardized test methods used to assess the engineering properties of fabrics, and a number of problems are encountered when test results obtained from different sources are compared. Also, project-life estimation may be practically impossible due to the difficulty in assessing and projecting long-term behavioral characteristics from short-term testing programs. A primary need exists for assessment of fabric properties required in field use and development of reliable laboratory tests for evaluating these properties.

Fabrics selected for an earth embankment or wall will require mechanical properties of high stress-strain modulus and low strain.

Where large strains are present in a soil-fabric system, such as riprap on fabric, a more extensible fabric that will not punch or tear is normally installed. Other factors that must be correlated with the mechanical properties of a fabric are the environment in which it is used and the manner in which it is placed. The ease of handling and workability of different fabrics in the field during construction may also determine which fabric is selected for a particular project.

Where fabric is used in such civil engineering applications as filters with water passing through the fabric, the drainage properties of the soil-fabric system must be thoroughly understood. Several design guidelines have been developed by different authors for the use of fabric as a replacement for soil filters.[4-8] These guidelines were based primarily on rules relating soil gradation to porosity and pore-size distribution of the fabric. There are, however, problems of reverse flow that can occur in tidal conditions, wave actions, or river stage fluctuations, such as rapid rise or drawdown. Reversing flow through the fabric may also occur beneath pavements or railway tracks subjected to rapid cyclic loading conditions that cause liquefaction of the foundation materials and consequent rapid rise in the hydrodynamic pore water pressure. These elements of behavior are very complex and this complexity opens to debate the success of a project, subject to dynamic loading, when the design is based on one-directional or nonreversing flow and quasi-static conditions.

Even when the engineer knows the physical conditions under which the fabric is to operate and has a general idea of the properties a fabric should possess, he may still be unable to make an intelligent selection from among various fabrics of different manufacturers. Nor is he likely to be able to specify desired properties and obtain a fabric with those

properties so that an efficient and economic structure is obtained. This is because there are currently no standardized tests by which to determine and compare the properties of fabrics for geotechnical use. Since most fabrics were initially developed for the textile industrial market, the tests developed in this area are of very little aid or interest to geotechnical engineers. Even though fabrics are produced under controlled conditions, there will always be some fluctuation in material properties that will require a series of tests to define the end-use potential of the fabric. Hopefully, in the future, tests will be developed to measure the fabric's fundamental engineering properties, so that comparisons among different fabrics can be made.

## Fabric Tests

### Permeability

The following paragraphs describe the laboratory and field tests reported in the literature.

Drainage and filtration properties of fabrics can be determined by laboratory methods designed to determine geotechnical properties of soils. These tests may be categorized as: (a) porosity and pore-size distribution of fabrics; (b) permeability of fabric across its plane; and (c) permeability of fabric along its plane (no standard tests exist for this category).

Woven fabric may be thought of as a sieve and the porosity and void distribution can be measured directly with a microscope or photographic enlargement of the fabric silhouette projected on a view screen.[9] A sieving technique is used for both woven and nonwoven fabrics because

the three-dimensional effects of the thick feltlike nonwoven fabrics do not provide a very open silhouette against a view screen. The sieving technique generally consists of vibration-sieving a range of single-sized granular particles for a suitable length of time and then determining the gradation curve that is equivalent to the fabric openings. Typical fabric pore-size distributions for woven and nonwoven fabrics vary from a medium silt to a coarse sand, but precise gradation for a particular fabric will depend on the methods and condition of testing. The gradations from woven fabrics normally resemble a coarse and uniformly graded sand material, whereas those of thick needle-punched and resin-impregnated fabrics range from a coarse silt to fine sand, with pore sizes less than 200 $\mu$ ($\mu = 10^{-6}$ in.).

Permeability across the fabric plane is easily determined in a constant head permeameter. Table III shows typical permeabilities of various types of fabrics.[10] Even though the test methods vary somewhat between laboratories, the test results appear to be comparable.

## Strength

Strength tests of fabrics often indicate significant differences. Strength tests may be categorized into three areas: (1) stretching the fabric in its plane; (2) deforming the fabric against its plane; and (3) tearing the fabric under intense localized shear loads.

Figure 4 shows some of the typical variations of tests that have been performed on fabrics to determine their strength properties.[11] From all the different tests shown in this figure, it is evident that there is no simple relationship among the results obtained from the various types of tests performed on a given fabric.

TABLE III

TYPICAL VALUES OF FABRIC PERMEABILITY
ACROSS THE FABRIC PLANE

| Fabric Type | Permeability, K, cm/sec |
|---|---|
| Woven | varies greatly >10 |
| Needle-punched | $10^{-1} - 10^{-2}$ |
| Melt-bonded | $10 \quad - 10^{-2}$ |
| Resin-bonded | $10^{-2} - 10^{-3}$ |

Simple tests that are designed for the expected field loads are most desirable; therefore, the strip tensile test, grab test, and plane strain tests are normally preferred. The plane strain test is not really that simple, but it is somewhat simpler and more economical than a sleeve test. The plane strain tests were developed because of neck-down characteristics of nonwoven fabrics in strip tensile tests.

Tests conducted by McGown on different types of fabrics using the plane strain test indicated that woven fabrics exhibit higher tensile strengths and lower strains, whereas the nonwoven fabrics have lower break strengths and higher extension to failure because of the way in which nonwovens are constructed.[10]

Nonwoven fabrics tested in a strip (uniaxial) tensile test will neck-down and progressively fail. The stray edges of the fabric cause considerable reduction in the fabric strength. Woven fabric in such a test does not neckdown and the test measures only the strength provided in fill or warp directions. Two-dimensional tests improve the performance of the

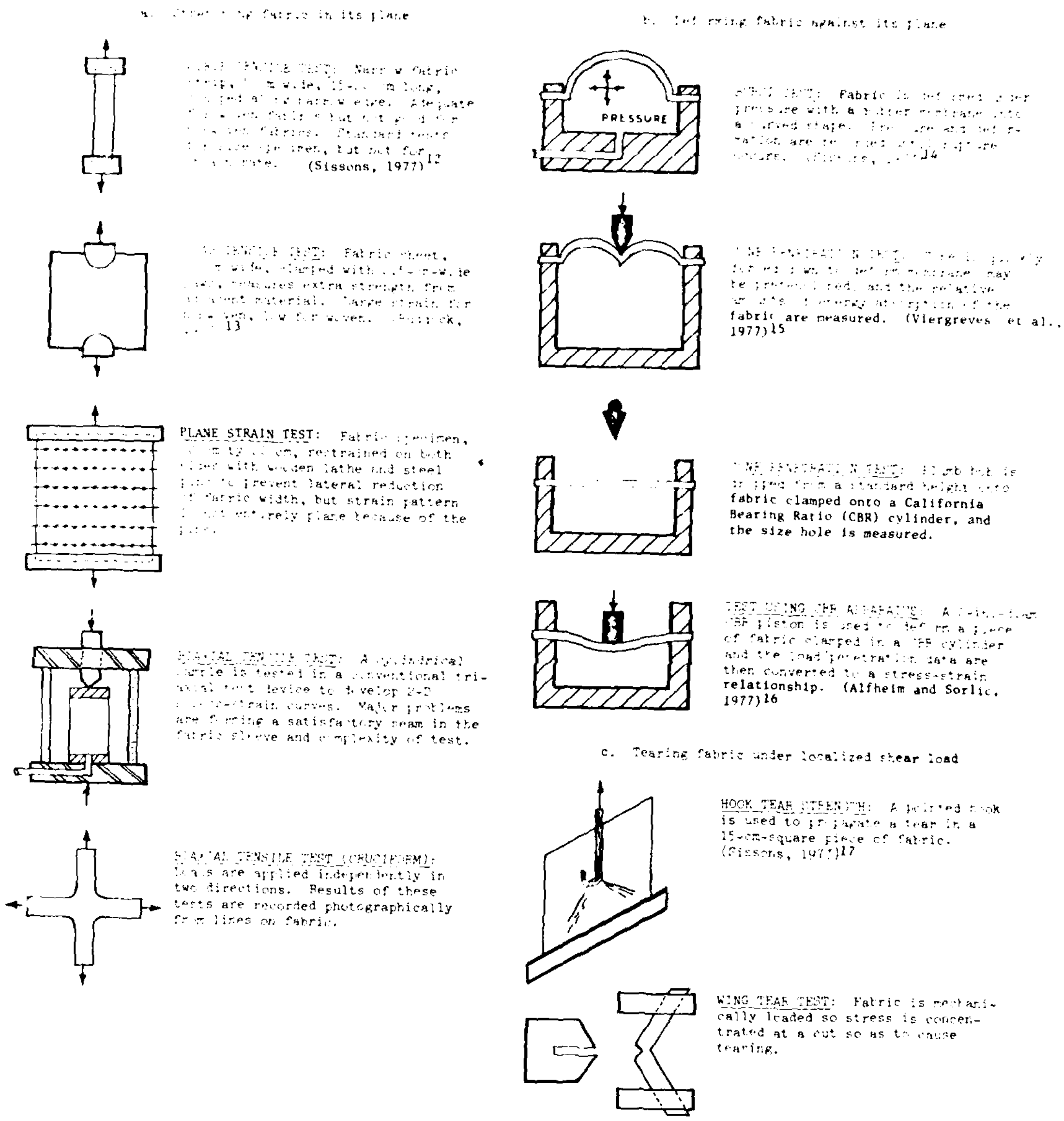



Figure 4. Typical Fabric Strength Tests[11]

nonwoven fabrics by about 30 to 50 percent, but since woven fabrics exhibit very little neckdown in uniaxial tension, there is little improvement of woven strength properties.[18]

Most nonwoven fabrics have high breaking strain and low stress-strain secant moduli, whereas woven fabrics have lower breaking strain and higher secant moduli. This indicates that the latter type would be better suited for reinforcement.

Usually laboratories apply the load to fabrics at a very rapid rate, and the test results may indicate a higher tensile strength than if the load were applied at a slower rate. This is a common error in load testing because inertial forces are measured by the load-sensing device ($F = ma$). Also, rapid testing may not allow complete realignment of nonwoven fibers prior to failure, biasing strengths on the high side.

Slow, sustained, and cyclic loadings of some polypropylene and polyethylene fabrics have shown varied results because of the visco-elastic creep properties of the fabric. Even when fabrics are made from identical polymers, the creep will depend on the factors related to the macrostructure of the fabric.

Other tests have been performed to investigate the damage susceptibility of fabrics subjected to falling pieces of riprap. Direct measurement of tear strength is given by the wing tear or hook tear tests, but other tests such as the cone penetration[19] and damage by aggregate tests[20] may be more representative of actual field conditions.

It has been noted in the past that the test strength of fabric may be affected by whether the fabric sample is tested wet or dry. Since the environment in most field conditions is wet and since the wet

strength is generally less than the dry strength, tests conducted after soaking each sample in water for 24 hr or more may be advisable.

A considerable need exists for standardized strength and permeability tests for all civil engineering uses of fabrics. No single test exists that will provide all the data necessary for the civil engineer to satisfactorily develop a project design incorporating geotechnical fabrics. Until more is known about the fabric properties that are predictive or descriptive of potential use in field applications, and until adequate laboratory tests are utilized or devised, it will remain difficult to select the most appropriate fabric for civil engineering projects.

## Field Tests

Synthetic fabrics have been used in filtration and drainage projects for the last decade: to replace one or more layers in graded sand/gravel filters, in erosion control projects where the fabric is protected from ultraviolet radiation by riprap or other materials, and to prevent piping or erosion of cohesive and noncohesive materials while allowing drainage and dissipation of pore water pressure. As a result of these particular applications, geotechnical fabrics are widely referred to as "filter cloth."

Most woven and nonwoven fabrics have an equivalent opening size (EOS) or porosity varying in a range comparable to U. S. No. 40 to 100 sieve openings. These fabrics are currently provided in 6- to 60-ft widths and in lengths of up to 5000 ft (on special order). Fabric costs vary from about $0.30 to $3.50 per square yard with the woven fabric

being generally more expensive, usually because woven fabrics are considerably heavier than the less expensive lightweight nonwovens.

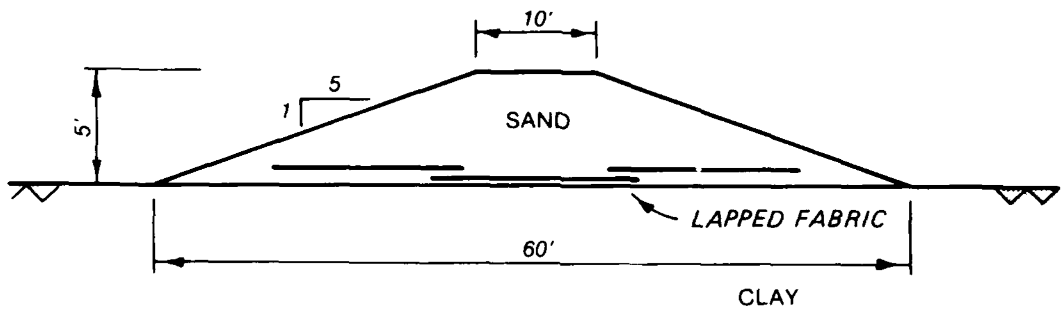
There have been only a limited number of fabric-reinforced embankment field tests conducted to evaluate the use of fabrics. Most of the fabric-reinforced sections that have been constructed were not built with testing in mind. Consequently, preconstruction and postconstruction exploration of foundation conditions was minimal and very little soil data were obtained.

Fabric-reinforced embankment sections constructed at Brunswick, Georgia, and at Swan Lake, Mississippi, will be discussed herein to show some of the problems that were experienced during design and construction.

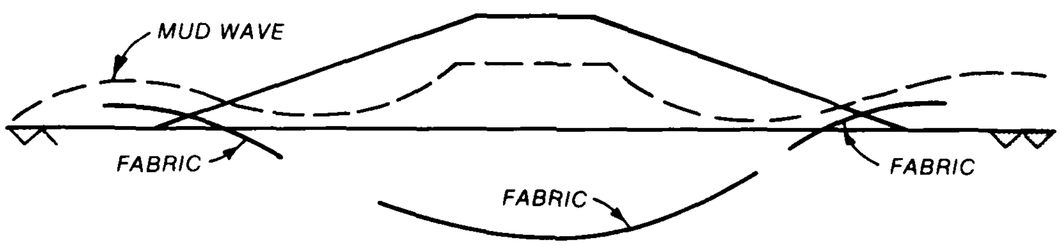
Brunswick, Georgia. A 3000-ft-long dredged material containment dike was to be constructed about 5 ft high and 60 ft wide across very soft foundation materials near Brunswick, Georgia. The structure was to be raised by end-dumping with single-axle dump trucks hauling sand from a nearby dredged material disposal area.

One 12-ft width of Dupont Typar 3401 (nonwoven heat-bonded polypropylene) was placed along the center line of the dike section over a sawgrass and weeded surface. Two additional widths of fabric were then placed parallel to the center line, overlapping the first strip by about 3 ft on either side (Figure 5a). As construction progressed, the embankment began to spread laterally and subside, moving the outside fabric sections with a mud wave (Figure 5b).

The project was continued by end-dumping displacement methods




a. DESIGN




b. AFTER CONSTRUCTION

Figure 5. Embankment Section, Brunswick, Georgia

until about 95 percent complete, when a catastrophic foundation failure occurred. Attempts to repair about 400 ft of the dike were unsuccessful, and projected costs for repair were deemed too excessive to complete the embankment.

The success or failure of this project was not determined by the fabric properties, but by the construction techniques employed to build the dike section. The fabric would have been more effective if it had been placed with the overlaps oriented perpendicular to the longitudinal axis of the dike. As a result of this fabric installation, it was learned that the fabric should always be oriented so that the seams are perpendicular to the longitudinal axis of the dike section, allowing the continuous fabric strip to resist the unbalanced loads.

Swan Lake, Mississippi. A 1600-ft-long test section was constructed at Swan Lake, Mississippi, in an attempt to determine the feasibility of constructing a fabric-reinforced embankment. This embankment would subsequently be used to protect a game reserve periodically flooded by water containing farming pesticides. Figure 6 shows the four 400-ft-long, 80-ft-wide test sections that were to be constructed to a height of 11 ft across an old oxbow lake. The lake had been filled with a deposit of very soft fat clay having an unconfined compressive strength of about 100 psf.

Test sections 1 and 3 were reinforced with Monsanto nonwoven fabric Bidim C-34 and Bidim C-28, respectively, and sections 2 and 4 were constructed without fabric. Each section was instrumented with

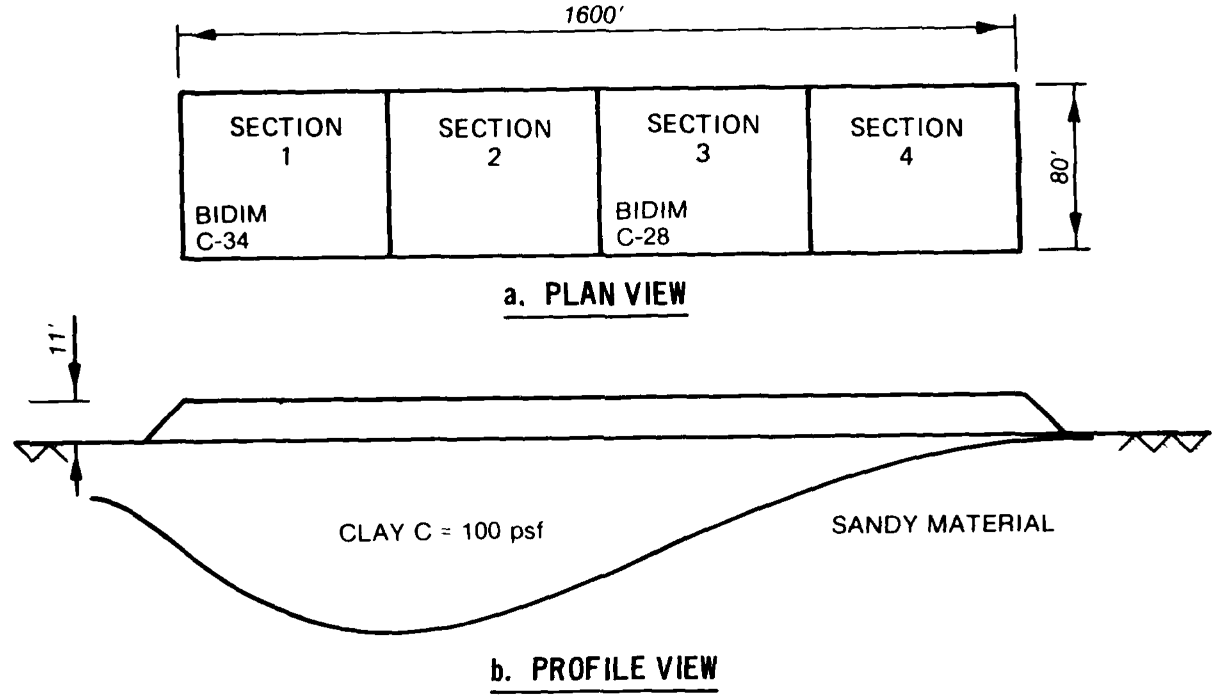



Figure 6. Plan and Profile View of Embankment Constructed at Swan Lake, Mississippi

vertical and horizontal slope inclinometer tubes and piezometers, which were monitored during construction.

Before the fabric was laid, trees on the heavily wooded side were cut down, delimbed, and covered with about 2 ft of lean clay material to form a working table. Once the fabric was positioned on the working table, the central longitudinal section was covered with about 1 ft of lean clay material. The exposed fabric edges were then folded back into the dike section to serve as an anchor to keep the fabric from slipping and the dike from spreading laterally (Figure 7a). All of the seams were sewn and the fabric oriented perpendicular to the longitudinal axis of the dike.

About 12 hr after the dike section was constructed to grade, it began to subside and spread laterally causing 6- to 12-in.-wide longitudinal cracks to appear along the crest and slopes, as shown in Figure 7b. The crest subsided about 3 to 4 ft and the depth of the cracks appeared to be 5 to 6 ft. Cracks were also observed in horizontal slope indicator pipes, which were subsequently abandoned when it became difficult to pass the inclinometer instrument through the pipes. Rather than attempt to repair the dike subsidence, continued construction was abandoned until the foundation materials consolidated and stabilized. It was concluded that excessive fabric elongation and low fabric modulus was responsible for the embankment failure.

## Need for Better Design Criteria

Figure 8 illustrates how the percent elongation of a fabric may be calculated when an embankment subsides under a triangularly distributed

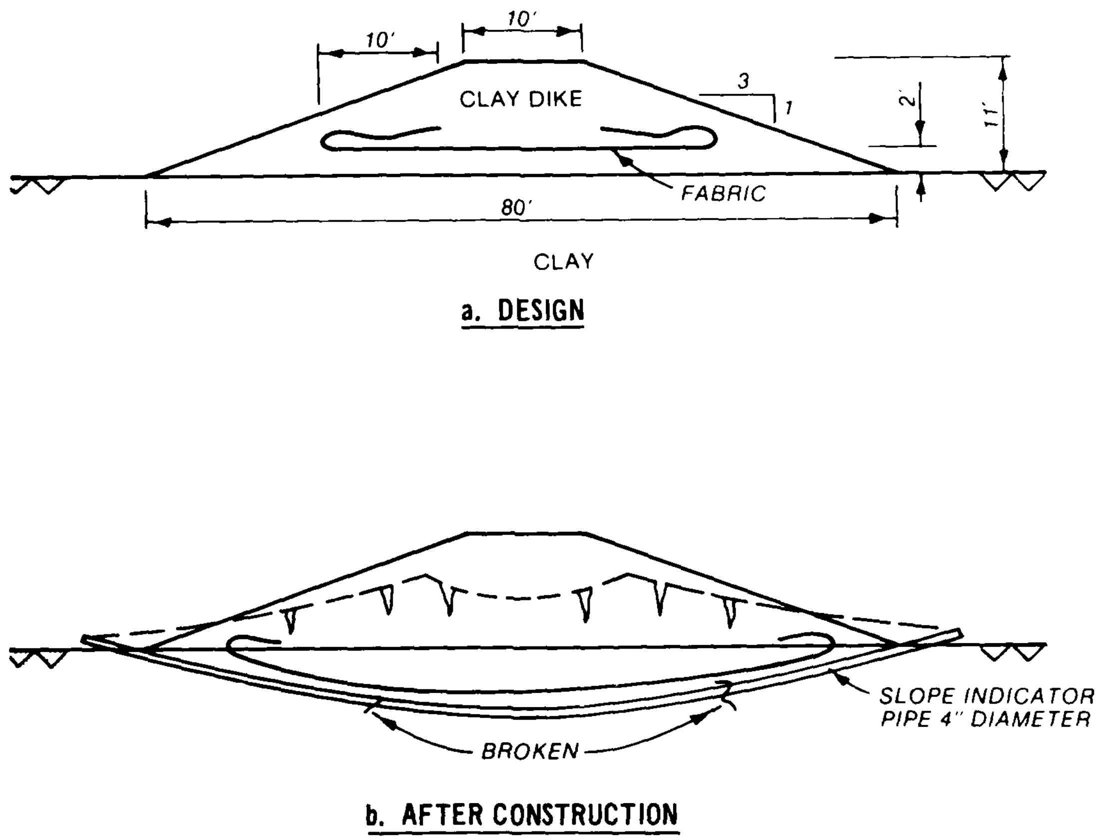



Figure 7. Embankment Section, Swan Lake, Mississippi

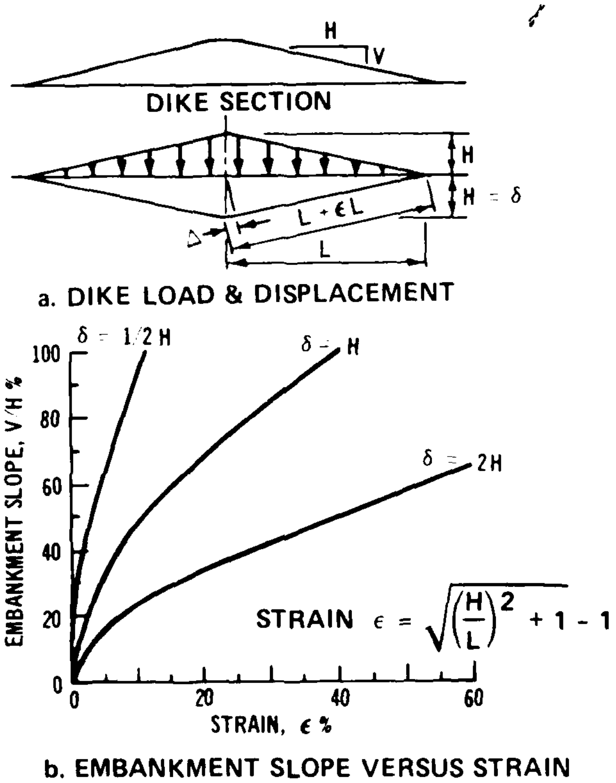



Figure 8. Embankment Slope versus Percent Fabric Elongation for Assumed Dike Displacements

load, assuming that no lateral displacement is allowed. To illustrate the percent fabric elongation for various embankment slopes, Figure 8b shows curves for assumed displacements expressed as a function of the embankment height H. It can be shown from Figure 8b that fabric elongation for assumed embankment displacements, H, increases as the slope increases. Even though the foundation displacement may not resemble a triangle but may be more parabolic in shape, Figure 8b demonstrates that fabric strains in an embankment must be very small if large deformation settlements are to be avoided. This suggests that a fabric with a high modulus of elasticity would be advantageous to resist large loads at relatively small strains.

Most fabric manufacturers' sales literature suggest the use of fabrics in roadways and embankments constructed on soft foundation materials and contain colorful photographs or artistic sketches of the finished structures. However, very little information on design criteria or the required geotechnical properties of the fabric and foundation material is given and disclaimers concerning technical reliability and manufacturers' liability for fabric use are always contained in the brochures.

A review of papers presented at a recent international conference held in Paris, France, on use of civil engineering fabric yielded several construction case histories of projects involving fabric-reinforced dikes, but few of these papers contained information on design and analysis.[21]

One paper presented at the 1978 American Society of Civil Engineers Geotechnical Engineering Specialty Conference on the Use of Solid Waste Materials described construction of a fabric-reinforced embankment

constructed of wood chips in Wisconsin.[22] However, the paper was concerned with reporting a case history of the project and little design and analysis data were presented.

A paper obtained through technical representatives of the Nicolon Corporation summarizes the results obtained from a fabric-reinforced dike constructed in Europe.[23] This paper presents the results of a consultant's study for Nicolon to develop design and analytic criteria for constructing fabric-reinforced dikes. Though this paper was very informative, it did not describe all methods of analysis needed for satisfactory embankment design.

In summary, it can be concluded that there is very little evidence of a substantial data base supporting the design and performance of fabric-reinforced earth embankments constructed on soft foundation materials. A definite need exists for standardization of fabric tests for civil engineering application and for correlation of these laboratory tests with data collected from actual field applications. To design and construct prototype embankments and to document the behavior of test structures is very important for successful implementation of this design concept.

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## CHAPTER IV

## DESIGN OF EXPERIMENTAL TEST SECTION

### Introduction

As already mentioned in Chapters I and II, previous studies supported by the CE Dredged Material Research Program (DMRP) and the MDO indicated that, to properly develop the Pinto Island disposal area, it would be necessary to construct dikes across the east and west ends of Pinto Pass and that a fabric-reinforced embankment would be the most cost-effective solution. Rather than enter immediately into full-scale construction, the decision was made to construct an experimental design test section at Pinto Pass.

Before construction of the test section could be initiated, however, it was necessary to establish criteria for the experimental design encompassing such items as location, size, construction procedure specifications for MDO contract advertisement, and numerous other related items and plans.

Since there were no engineering test standards for comparing the merits of various geotechnical fabrics for use in embankment reinforcement, the first priority of the experimental design was establishment of a method for fabric selection. Therefore, a study was conducted to obtain, test, and evaluate currently available civil engineering fabrics for use as embankment reinforcement.[1] These data were used in the subsequent design of the fabric-reinforced embankment test section

across the west end of Pinto Pass, Mobile Harbor, Alabama.[2]

## Design Constraints

The design constraints for the fabric-reinforced embankment, described in detail in Haliburton et al. (1978),[1] indicated that the embankment was to act as a multipurpose structure to:

1. Allow initial containment of dredged material up to el 8.
2. Act as a preload structure to consolidate underlying soft foundation materials and to allow rapid dike raising to at least el 25.
3. Provide a wide stable base section for future dike raising.[3]

The embankment test section was to be located along the proposed dike alignment to minimize the total dike construction cost. In the event the test section construction was successful, it could be incorporated into the disposal area containment dike system. The dike was to be initially constructed to el 8 and raised to el 12 with coarse-grained material available from nearby dredged material containment areas. Subsequent raising would be conducted with dewatered fine-grained dredged material from inside the Pinto Island disposal site. These constraints resulted in the selection of an initial embankment section with a crest of el 8, a 12-ft crest width, and 1V on 10H side slopes. This initial embankment section would provide a stable base section for raising to el 25 with 1V on 3H side slopes and would allow for future raising to el 50 in the event it was required by the MDO.

The north-to-south embankment test section was also constrained at the west end of Pinto Pass by the following:

1. Existing dikes at about el 12 to 16 located on the north abutment that would eventually be raised and renovated during the overall Pinto Island disposal site dike construction.

2. The need to locate the dike alignment as far east of an existing bridge as possible without causing undue loss of potential disposal storage volume but far enough to minimize disturbance to this structure in the event of test section failure.

3. The need to locate the south end of the test section 400 ft north of the center line of a paved access road to the Alabama Dry Dock and Shipbuilding Company (ADDSCO).

The test section embankment was located as shown in Figure 9;[2] the probable future dike alignment is also shown in this figure. A larger scale plan view of the embankment is shown in Figure 10.[2]

## Design and Construction Considerations

The most important design consideration for test section construction was the existing foundation profile across the west end of Pinto Pass. As a result of limited exploration by the Core Drill Section, MDO, and more detailed exploration, sampling, and testing conducted by the U. S. Army Engineer Waterways Experiment Station (WES), it was determined that extremely soft foundation conditions existed across the east and west ends and along the south tidal line of the pass.

Surface elevations for both ends of the pass varied from about el 1.0 to el -1.5. Below the surface, very soft organic clays and silty clays with interbedded thin layers of sand existed down to a dense sand at about el -40. For design purposes, field vane shear tests determined

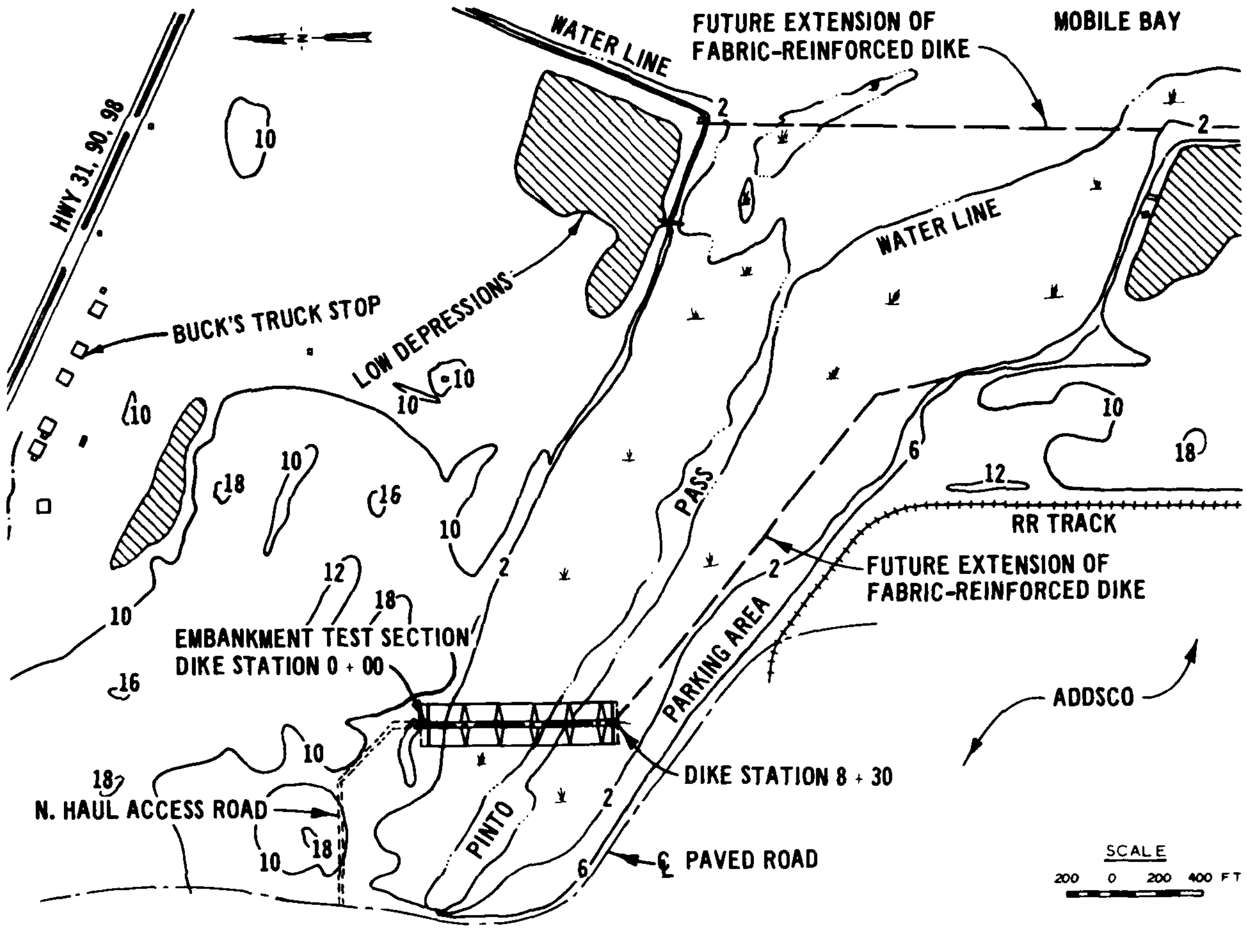



Figure 9. Location of Embankment Test Section, Pinto Pass, Mobile, Alabama[2]

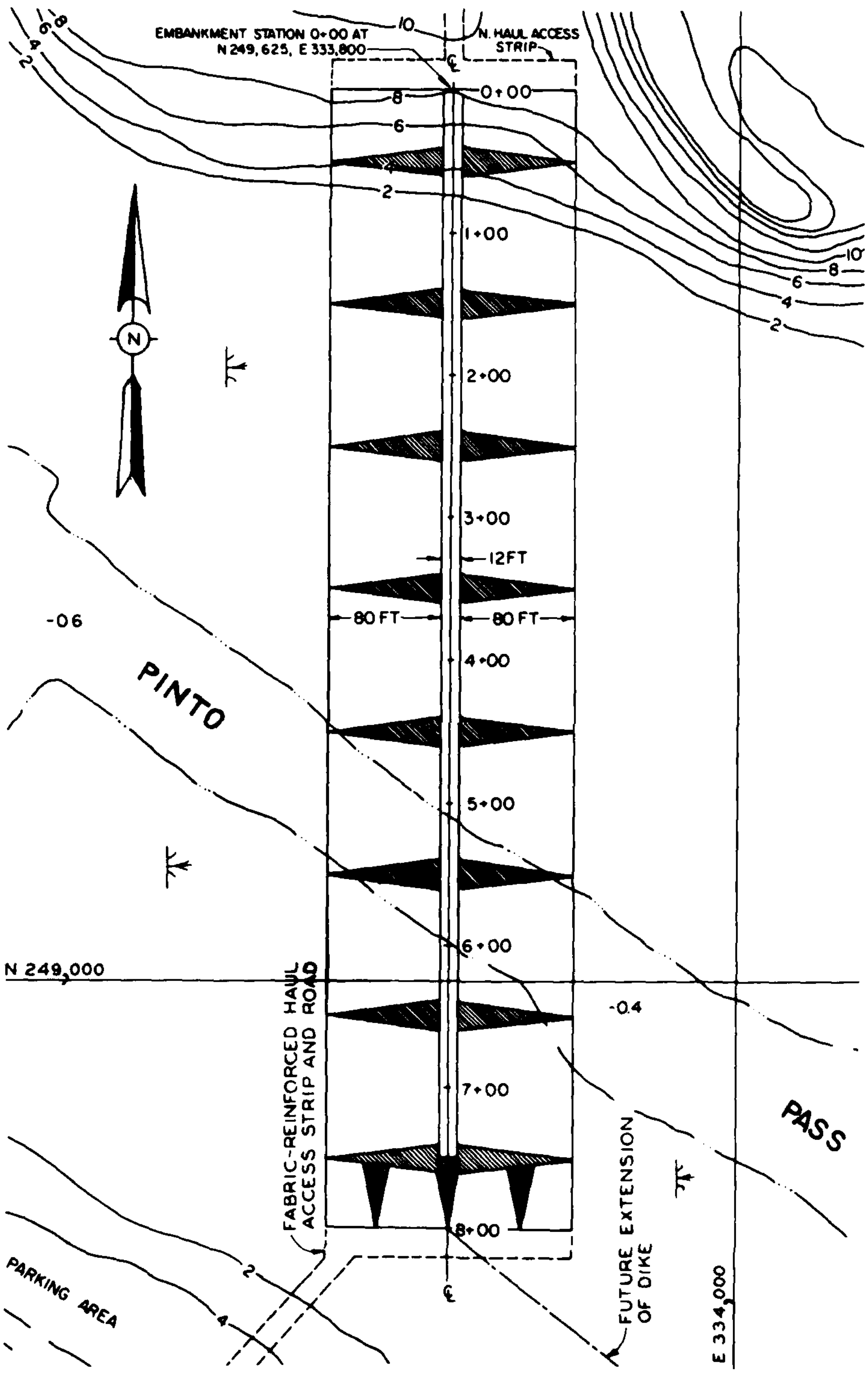



Figure 10. Detailed Plan View of Embankment Test Section[2]

that the upper 5-ft stratum had an average cohesion c of 50 psf underlain by another 10-ft stratum with cohesion c equal to 100 psf, which was underlain by approximately 25 ft of material with cohesion c equal to 150 psf. The profile along the south side of the pass was similar except that the average cohesion c was equal to 100 psf in the upper 15 ft and the material contained more sand lenses.

Assuming that the dike would be constructed to el 8 using fine- to coarse-grained sand from a nearby dredged material containment area, the maximum (center line) dike-bearing pressure would be approximately 1000 psf. Ultimate bearing capacities for the soft structure underlying the center of the pass would be approximately 300 psf for the $c = 50$ psf material, 600 psf for the $c = 100$ psf material, and 900 psf for the $c = 150$ psf material. From these data, it can easily be seen that the design problem was one of providing adequate bearing capacity since the bearing pressure exceeded the available foundation bearing capacity.

Constructing a dike by normal procedures of a displacement section would have resulted in a bearing failure once the dike reached a height of about 3 ft or the dike plus the construction equipment bearing pressure exceeded 300 psf. Remolding the clay foundation materials with the construction equipment might have reduced the bearing capacity of the foundation materials even further. Unless adequate bearing capacity could be provided, analyses for slope stability and potential consolidation settlement would have been meaningless. Since the dike might ultimately be raised incrementally to el 50 ft, any design should allow for bearing pressure of about 6000 psf to avoid bearing failure.

## Possible Dike Designs

Foundation conditions such as those previously described are generally preloaded to allow consolidation settlement to densify the soft underlying material and increase the bearing capacity until the design load can be supported without bearing failure. In this case, foundation conditions indicated that a dike height of 3 ft might be achieved if construction equipment did not further destroy the foundation material bearing capacity. Without careful field control, the most probable engineering result of any construction would be a displacement section.

Advancement of dike sections across and through soft foundation materials by end-dumping and displacement, similar to the techniques shown in Figure 11, is a commonly accepted construction technique of many CE Districts.[4,5] Dike design alternatives (Figure 12) that were evaluated included use of sand berms, dike construction with lightweight materials, and partial to total excavation and replacement of the soft foundation materials. Of the design concepts considered, only two were found to offer potential technical success: advancement of the fill using the end-dumping and displacement technique, or construction of a "floating dike section" using fabric as tensile reinforcement to carry the excess bearing pressure.[6,7,8]

After careful consideration of the above alternatives, it was decided that the floating section would be the most practical since the displacement technique could result in mud waves and foundation displacement with possible disruption or damage to abutting structures (a paved road and a bridge along the west edge and utility right-of-way

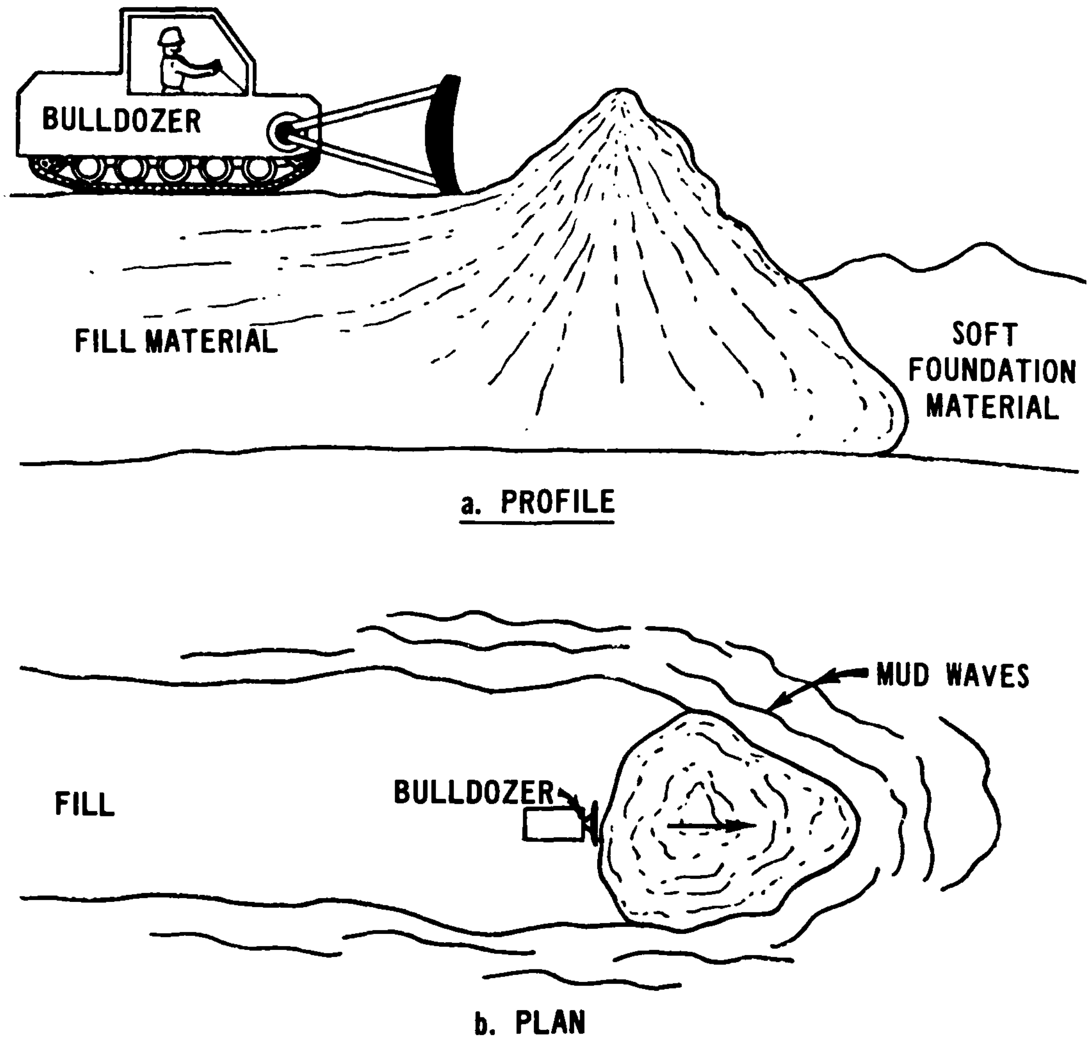



Figure 11. Advancement of Fill Using End Dumping and Displacement Technique[4]

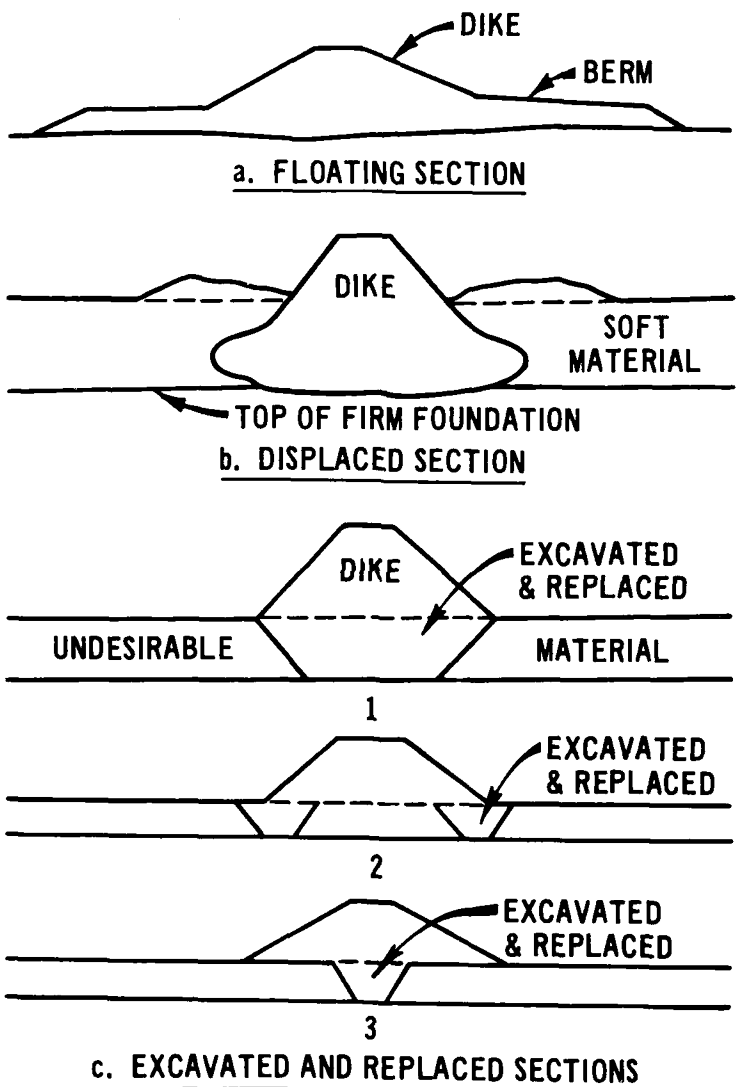



Figure 12. Basic Dike Concepts[4]

and parking facilities along the south shore of the pass); additional cost compared with that of the floating dike design; and decreased potential for effective construction control in the field.

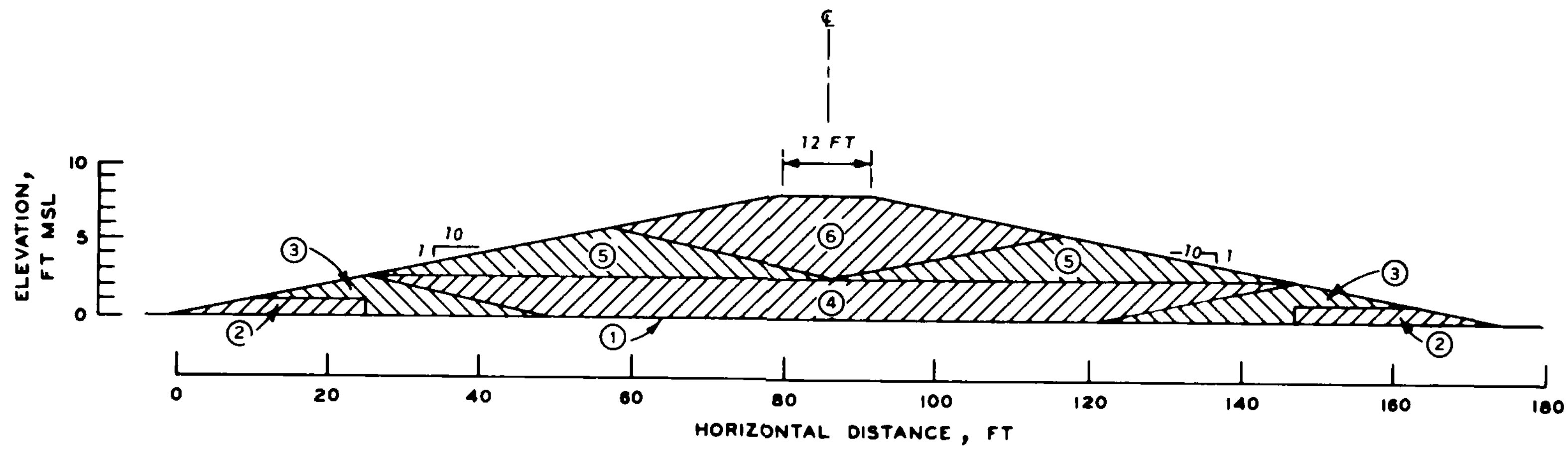
## Proposed Test Section Design

Since there were two different designs proposed for the three dikes at Pinto Pass and since the long south shoreline dike involved a less complex and difficult type of design, it was decided that the most data could be generated in the least amount of time by initiating construction on the short, more complex design dike at the west end of Pinto Pass. The dike was to be constructed of fine- to coarse-grained dredged material sand borrowed from nearby dredged material disposal sites and would have a 12-ft crest at el 8 and 1V on 10H side slopes. Fabric was to be employed at the base of the dike section. The wide dike would provide a preload consolidation pressure over a wide area to increase the strength of the soft foundation materials, plus a stable base section for future dike raising.

Sequential construction, shown in Figure 13, is probably the most important factor in obtaining satisfactory performance of any fabric-reinforced embankment. The construction sequence proposed for the test section is summarized as follows:

1. Fabric was to be laid on the surface in continuous transverse strips over a thin (1-ft) sand working table with approximately 20-ft overlap or excess at each end and with adjacent transverse strips slightly overlapped and sewn together.

2. During placement of the fabric, two outside access and anchorage strips were to be constructed by covering the fabric with about




SEQUENCE OF CONSTRUCTION

1. LAY FILTER CLOTH IN CONTINUOUS TRANSVERSE STRIPS, SEW STRIPS TOGETHER.
2. END-DUMP ACCESS ROADS AND LAP FILTER CLOTH OVER TOP.
3. CONSTRUCT OUTSIDE SECTIONS TO ANCHOR AND STRETCH FILTER CLOTH.
4. CONSTRUCT INTERIOR SECTION TO ANCHOR FILTER CLOTH.
5. CONSTRUCT INTERMEDIATE SECTIONS TO TENSION FILTER CLOTH IN CENTER.
6. CONSTRUCT FINAL CENTER SECTION.

Figure 13. Cross Section and Construction Sequence for Proposed Floating Dike to el 8 Across East and West Ends of Pinto Pass[9]

1 ft of fill material. These access strips were to be carried as far as possible with the excess fabric at each end lapped and buried before the next operation was started.

3. Two small outside dikes were then to be constructed to anchor the fabric and the resulting vertical settlement under these dikes would stretch the fabric in the center of the dike.

4. The center section would then be filled to anchor the fabric along the entire transverse length of the dike section.

5. Intermediate dike sections would then be constructed to cause settlement toward the outside of the dike, again creating tension in the fabric near the dike center.

6. Finally, the center section would be constructed to design el 8.

When the fabric settled or deformed as a result of the overlying sand compressing the foundation, it was anticipated that the fabric would develop tensile stresses that would counteract the forces from the sand weight and thereby prevent bearing failure deformation and reduce the net foundation contact pressure. It was also anticipated that internal displacements in the dike sand material would cause internal arching that would serve to transmit vertical stresses from the center of the dike section toward the outside edges of the dike, where the foundation contact pressure would be less. This would develop a more uniform distribution of the bearing stresses across the dike. This same behavior might also occur longitudinally along the dike center line, causing further fabric tensioning. It was originally postulated that, if the construction sequences were not followed carefully and the fabric was not anchored properly, the fabric might not carry the dike loadings necessary to prevent excessive deformation.

## Potential Embankment Failure Modes

To design a dike for successful function both during and after construction, with only limited information on the behavior of fabric-reinforced embankments, it was necessary to investigate four failure modes that might occur: (1) sliding wedge failure of the embankment; (2) local bearing failure of the soft foundation; (3) failure by excessive settlement before stable bearing conditions were achieved; and (4) insufficient fabric anchorage during embankment deformation.

### Sliding Wedge Failure

As shown in Figure 14a, a sliding wedge failure could occur by lateral outward displacement of the embankment, essentially by sliding along the underlying fabric layer. Assuming the height of the embankment is fixed by other constraints, controlling parameters in wedge sliding stability would appear to be the embankment side slope angle and the coefficient of sliding friction between the embankment material and the fabric. Soil-fabric properties would require that the coefficient of soil-fabric friction be equal to or greater than the equivalent soil friction for the embankment material. Therefore, if the soil-fabric frictional behavior is known, the embankment side slopes required to achieve necessary wedge sliding stability could be determined.

### Local Bearing Failure

Local bearing failure of the soft foundation, illustrated in Figure 14b, is the result of a rotational/slumping failure of part of the embankment. Assuming a side slope is chosen that would satisfy

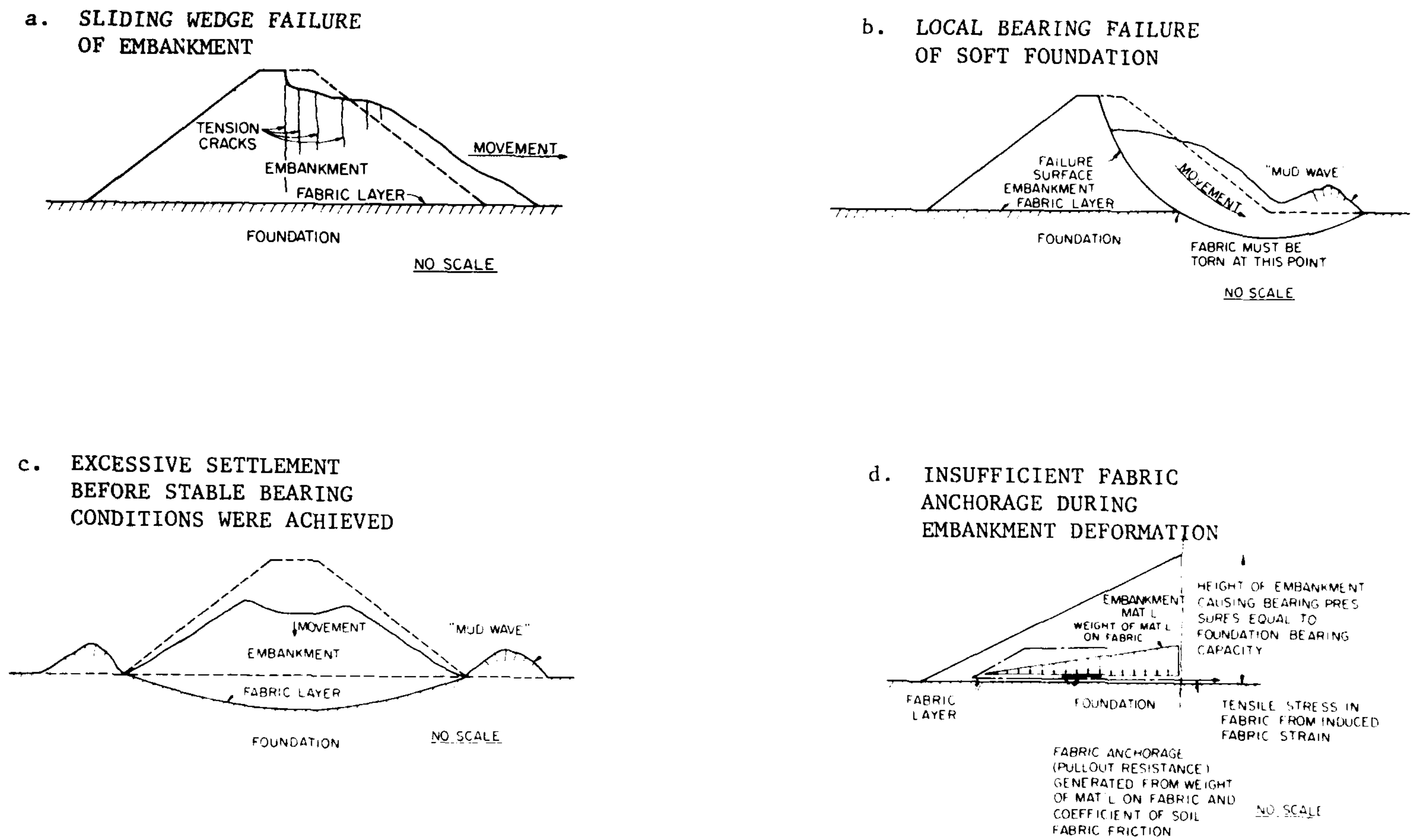



Figure 14. Potential Failure Mode that Might Occur for Fabric-Reinforced Embankments[1]

internal embankment slope stability requirements and prevent wedge sliding along the embankment-fabric interface, weight of the embankment could still trigger a rotational-type foundation failure extending through a portion of the embankment. For this type of failure to occur, the fabric layer must fail in tension and not in the anticipated mode because the fabric has no flexual strength to resist shear forces. The fabric's ultimate tensile strength would be mobilized prior to embankment rotation failure; therefore, a conventional slope stability analysis could be made if the ultimate tensile strength of the fabric were added to the available shear strength of the soil. The design procedure to determine stability would be to adjust the side slope (and thus the resisting forces, including fabric ultimate tensile strength) to match foundation soil strength, with a suitable factor of safety.

## Failure by Excessive Settlement

Embankment failure could occur by excessive settlement before stable bearing conditions are achieved, as shown in Figure 14c. A fabric with low tensile modulus would stretch excessively under imposed embankment weight and the resulting large settlements could render the embankment useless for its intended purpose. This requirement (low fabric deformation modulus) is not considered in other analyses where fabric ultimate strength controls. The fabric must support the difference between the ultimate bearing capacity of the foundation and the bearing pressure of the embankment.

To obtain desired fabric behavior, it was suggested that sequential construction of the embankment might be required.[9] To develop and maintain optimal fabric support for the center of the dike, it was

proposed that the outside portions of the embankment, near the toe, be constructed first to provide anchorage. Then as the center section was constructed and the bearing pressure of the embankment exceeded the bearing capacity of the foundation, deformation would occur in the center portion of the embankment and create strain in the fabric. A stable condition was anticipated when stresses created by the induced strain from fill from the outside portions were great enough to carry the increased embankment bearing pressure when the center portion was filled.

A design to prevent embankment failure can be determined from the stress-strain behavior of a given fabric. The difference between the foundation-bearing capacity and embankment bearing pressure would be the equivalent fabric stress. If the stress-strain properties of the fabric were known, then deformation of the embankment could be determined and a fabric could be chosen to meet or exceed the stress-strain criteria for a specific project. Initially, it was arbitrarily assumed that average fabric elongation would be on the order of 3 to 5 percent and that localized strains would not exceed 10 percent.

Insufficient Anchorage

Insufficient anchorage of fabric ends in the toe of the embankment might allow fabric slippage during embankment deformation and result in excessive center line embankment settlement. Sequential construction of the embankment as described in the preceding section would require that the fabric be folded back into the dike section as shown in Figure 14d and the weight of material overlying the fabric in these zones must produce enough anchorage to prevent fabric slippage. The critical

design parameters for this condition would appear to be (1) the embankment side slope; (2) the weight of material outside the zone of expected foundation bearing failure; and (3) the coefficient of friction between the fabric and embankment material.

### Possible Effect of Various Failure Modes on Actual Embankment Deformation

It should be noted that unsatisfactory behavior as defined in Figures 14a and 14b tends to cause outward movement of the embankment, while the unsatisfactory behavior defined in Figures 14c and 14d would tend to allow inward/downward movement of the embankment. Thus, during actual construction, the embankment deformations may be reduced because of a tendency for opposing effects to cancel each other, producing more nearly uniform displacements.

## Fabric Design Criteria

From the above potential embankment failure modes, the design properties of a fabric needed for a reinforced embankment can be identified. Haliburton, Anglin, and Lawmaster concluded that these properties were fabric stress-strain behavior, ultimate tensile strength, soil-fabric frictional resistance, creep resistance, and wet strength.[10] The most desirable fabric would be one that had high elastic modulus, high tensile strength, ability to undergo large deformations without rupture, and negligible creep under working load, i.e., the properties of mild steel, plus corrosion resistance.

To prevent local foundation bearing failure and/or embankment rotational failure, high ultimate tensile strength fabric is required to

resist the unbalanced loads that occur at right angles to the longitudinal axis of the dike. Since the fabric reinforcement would be composed of relatively long and narrow strips, the fabric was considered to be in uniaxial tension when ultimate strength was developed.

Biaxial load testing was not considered appropriate because compression loading of a soil-fabric system does not stress the fabric. Forces parallel to the embankment alignment were considered to be balanced whereas the unbalanced forces that would cause fabric deformation were perpendicular to the alignment. Therefore, uniaxial tension tests to determine the stress-strain behavior and ultimate tensile strength properties were required.

Displacement of embankment material (sand) against a fabric under various values of applied normal stress could be determined in a direct shear device that had previously been used to determine soil-soil frictional properties for the embankment material.

Fabric creep, the tendency of a fabric to elongate under a static load with time, was also determined at given fabric design working stresses. It was considered desirable to select a fabric with relatively low creep properties under design stress levels.

Most synthetic fabrics have relatively high resistance to corrosion, bacterial action, and other effects, and some degree of resistance to ultraviolet radiation (sunlight); but, since the fabric at Pinto Pass would be buried in the intertidal zone and continuously immersed in salt water, tests were conducted to compare saltwater-soaked tensile strengths to those in an unsoaked condition.

## Fabric Tests

The 27 commercially available geotechnical fabrics that were tested were composed of various combinations of polypropylene, polyamide (nylon), polyesters, and polyolefin. Of the 27 fabrics tested, there were 16 nonwoven fabrics, 10 woven fabrics, and 1 combination fabric (woven and nonwoven). In addition to the 27 petrochemical-based fabrics, one fiberglass fabric provided by Bay Mills Midland, Ltd., of Midland, Ontario, Canada, was tested.

All the fabrics were subjected to uniaxial tension tests to determine the stress-strain characteristics of each fabric, including ultimate tensile strength and stress-strain modulus. Previously established design criteria for the Pinto Pass embankment test design called for a minimum strength of 100 lb/in.-width tensile stress, at 10 percent strain.[11] Fabrics meeting or exceeding the tensile strength criteria were subjected to further testing. These tests were creep measurement, soil-fabric friction resistance by direct shear, and tensile strength after soaking in artificial seawater for 5 weeks.

## Test Results

Only four woven petrochemical fabrics met or exceeded the criteria of 100 lb/in.-width tensile stress at 10 percent strain.

The test data indicated a fairly wide variation in the tension stress-strain behavior of the 27 geotechnical fabrics and 1 fiberglass fabric tested. It was determined that woven fabrics were considerably stronger than nonwoven fabrics in uniaxial tension. The woven fabrics failed from localized strand breaking whereas the nonwoven usually

failed by excessive elongation or lateral neckdown, with Poisson's ratio exceeding the theoretical maximum for a uniform material.

The four woven petrochemical fabrics that were found to exceed 100 lb/in.-width stress at 10 percent strain criteria also had a considerably higher ultimate tensile strength and stress-strain modulus than all the other fabrics tested. These were Nicolon 66475, Polyfilter X, Advance Type I, and Nicolon 66186. The highest stress-strain modulus among all fabrics tested was for Bay Mills 196-380-000 woven fiberglass. Consequently, although the fabric failed by tearing at 8 percent strain, it was included in the test program for comparative purposes. A comparative plot of stress-strain data in the warp direction for the five fabrics is shown in Figure 15. These fabrics were then subjected to testing for creep behavior, soil-fabric frictional resistance, and effects of saltwater soaking on tensile strength.

Creep tests indicated that, of the five fabrics tested, Bay Mills 196-380-000 had essentially zero creep; Nicolon 66475 and Nicolon 66186 had essentially minimal creep; Polyfilter X had moderate to high creep tendencies; and Advance Type I had high to extremely high creep tendencies. The results of creep tests are shown in Figure 16.

The friction angle, $\phi$, between Mobile sand and the five woven fabrics tested indicated that the results were about the same as the sand alone friction angle for the sand in a loose relative density condition and were several degrees less than soil alone friction angle for the sand in a dense relative density condition. Therefore, for design purposes, the friction angle $\phi = 30^{\circ}$ for the soil alone in a loose relative density condition was considered to be satisfactory.

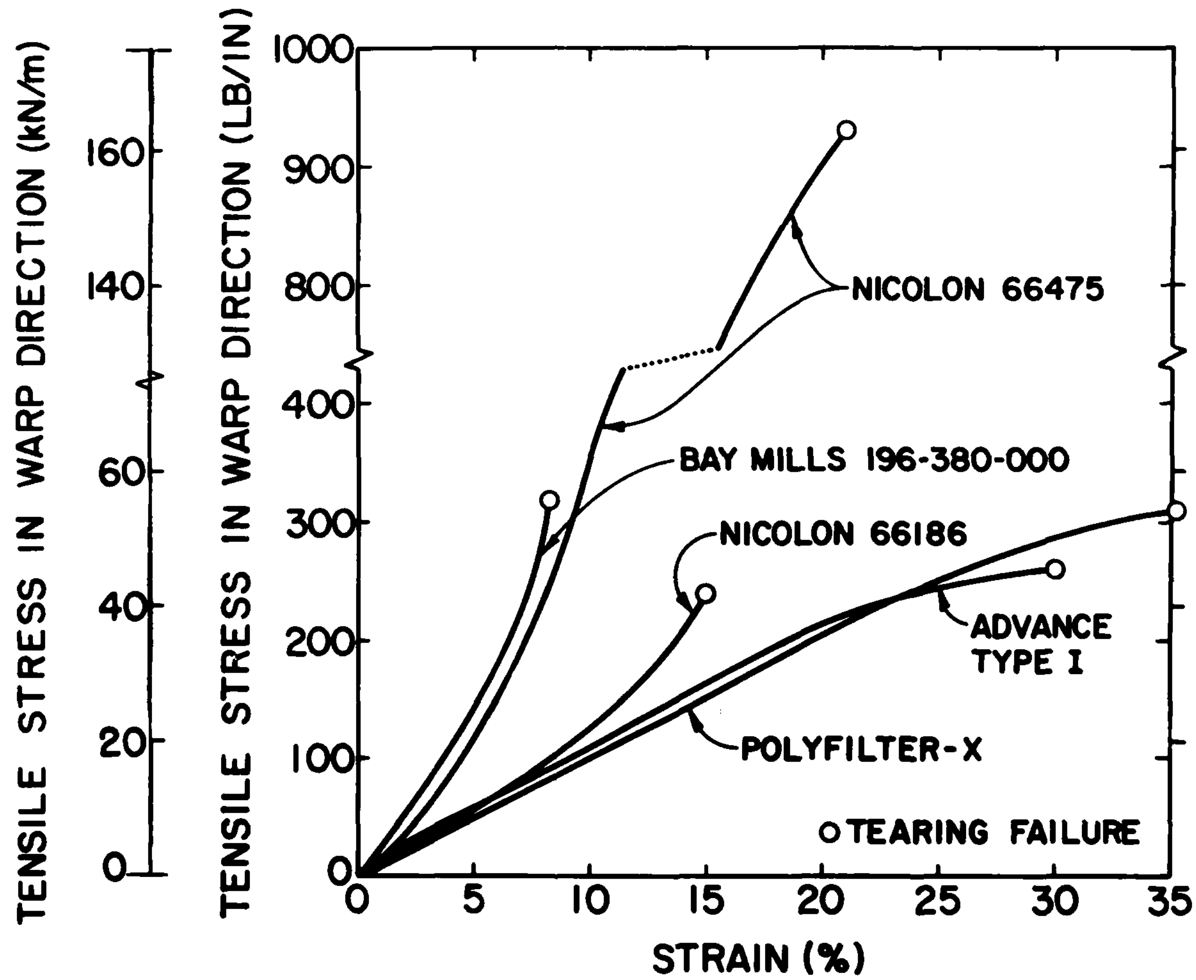



Figure 15. Stress-Strain Data for the Five Geotechnical Fabrics Meeting Desired Tensile Strength Criteria[1]

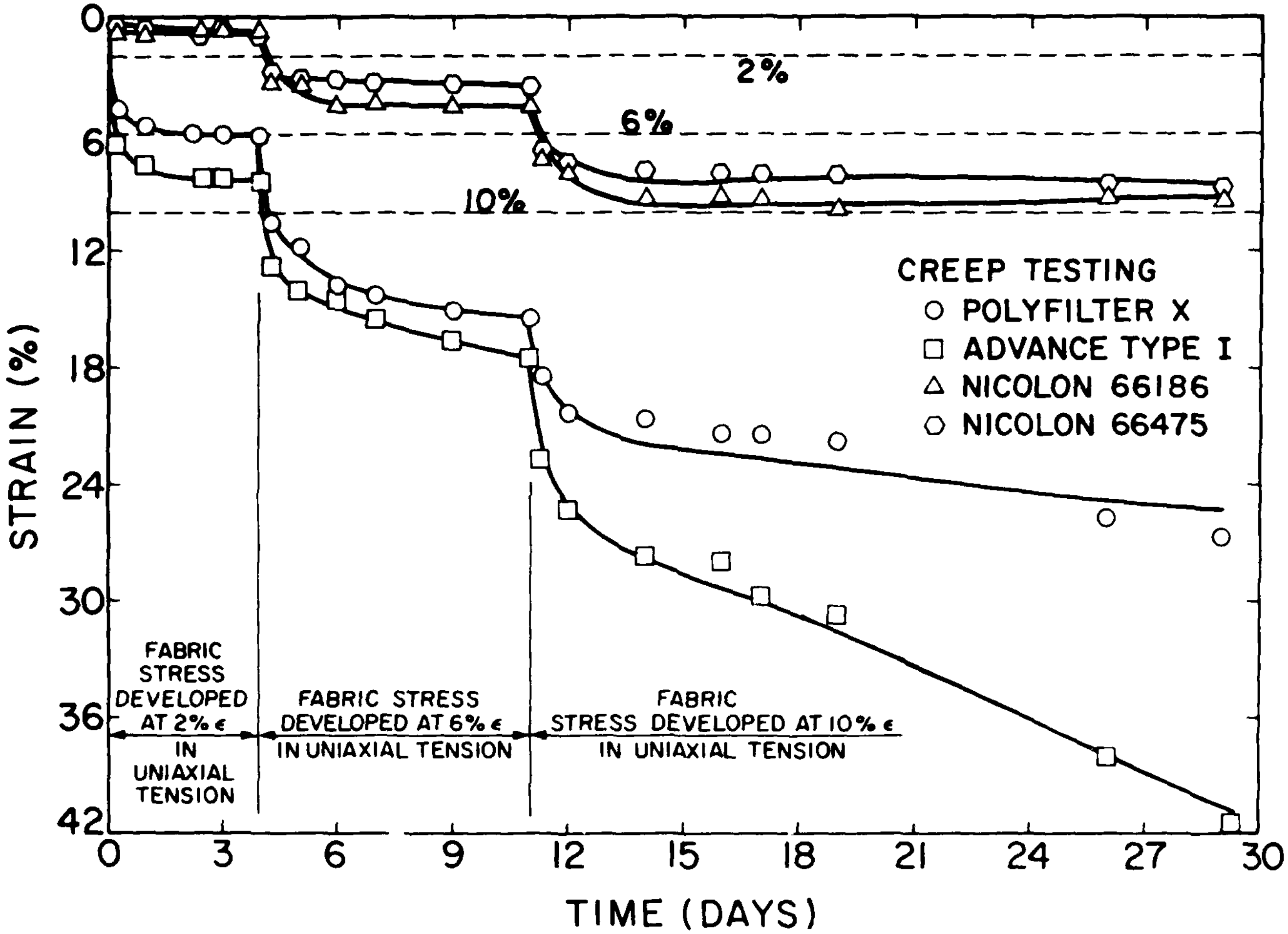



Figure 16. Results of Geotechnical Fabric Creep Testing[1]

Uniaxial tests conducted after 5 weeks of soaking in saltwater indicated that Nicolon 66475 and Nicolon 66175 had negligible strength loss, Advance Type I had an 18 percent strength loss, and Polyfilter X had a 32 percent strength loss. Bay Mills fiberglass was not tested because of delayed acquisition of fabric sample.

Therefore, it was recommended that, because of their high resistance to creep and minimal strength loss due to soaking, Nicolon 66186 and Nicolon 66475 be used in the portions of the test section where maximum fabric stress levels might occur. For evaluative purposes, the Advance Type I and Polyfilter X were recommended for portions of the dike test section where less than maximum stress was expected. The Bay Mills fiberglass could be used if further testing does not indicate loss of strength when wetted. The results of the tests conducted and values used in the design of the test section are shown in Table IV.

## Embankment Analysis

A detailed study was conducted to determine the applicability of current structural mechanics theories of membrane, thin-plate, and thin-shell behavior to the problem of analyzing a fabric-reinforced embankment on soft foundation.[12] It was decided that these theories were not applicable because they did not adequately consider foundation support characteristics, required the assumption of permanent fixed anchorage of the fabric, and did not consider the effect of internal embankment arching and load redistribution by soil displacement.

It was assumed that the loading of the long fabric-reinforcement strips placed transverse to the dike alignment would be in uniaxial tension and the membrane-oriented theories assume biaxial stress

TABLE IV

RESULTS OF FABRIC TENSION TESTING[1]

| Fabric Trade Name or Manufacturer Designation | Woven (W) or Nonwoven (N) | Warp (W) or Fill (F) | Tensile Stress (lb/in.) @ 5%ε | 10%ε | Ult. | Strain @ Stress T = torn E = 50% elongation S = strength drop (%) | Soaked Ultimate Strength (lb/in.-width) | Initial Tangent Modulus $E_i$ (lb/in.-width) | Secant Modulus $E_s$ at 10%ε (lb/in.-width) |
|---|---|---|---|---|---|---|---|---|---|
| Nicolon 66475 | W | W | 110.6 | 361.7 | 903.3 | 21T | 845 | 714 | 3620 |
| | | F | 40.6 | 107.0 | 159.7 | 15T | | 577 | 1070 |
| Polyfilter X | W | W | 52.7 | 102.8 | 311.3 | 35T | 212 | 1429 | 1028 |
| | | F | 31.1 | 65.7 | 184.2 | 33T | | 564 | 657 |
| Advance Type I | W | W | 57.7 | 107.5 | 251.5 | 29T | 207 | 3500 | 1075 |
| | | F | 25.6 | 50.5 | 137.2 | 29T | | 697 | 505 |
| Nicolon 66186 | W | W | 46.1 | 108.5 | 226.0 | 15T | 208 | 260 | 1085 |
| | | F | 56.9 | 130.8 | 241.8 | 15T | | 1000 | 1038 |

conditions. Also the use of a membrane supported by elastic springs was considered, but there were no known computer programs to solve a statically indeterminate problem in soil-structure interaction similar to this problem and it was considered to be beyond the scope of the preliminary design study to locate or formulate a program of this nature.

The final conclusion was that, to properly design the test section, a finite element modeling technique would have been more appropriate, but there were too many unknowns to allow an accurate before-the-fact prediction of behavior. It was decided that use of the finite element technique after construction would be more proper.

As a result of nonavailability of more sophisticated methods, a simpler approach to design was attempted based on resistance to the four unsatisfactory modes of potential behavior for civil engineering *fabric-reinforced embankments* on soft foundation shown previously in Figure 14.

The embankment cross section used for design purposes is shown in Figure 17. The major difference between the section chosen for analysis and the typical construction cross section would be that the fabric is assumed to be located at the base of the 8-ft embankment when in actuality it may be located at el 1.0 to 1.5. This difference would not affect performance because the effective depth of the dike would be smaller.

In addition to the assumed embankment design cross section, the following detailed data and/or assumptions were made for the analysis:

1. Maximum center line settlements were computed by Haliburton, Douglas, and Fowler from consolidation under initial construction and successive dike raising.[13] Settlements were computed assuming normally

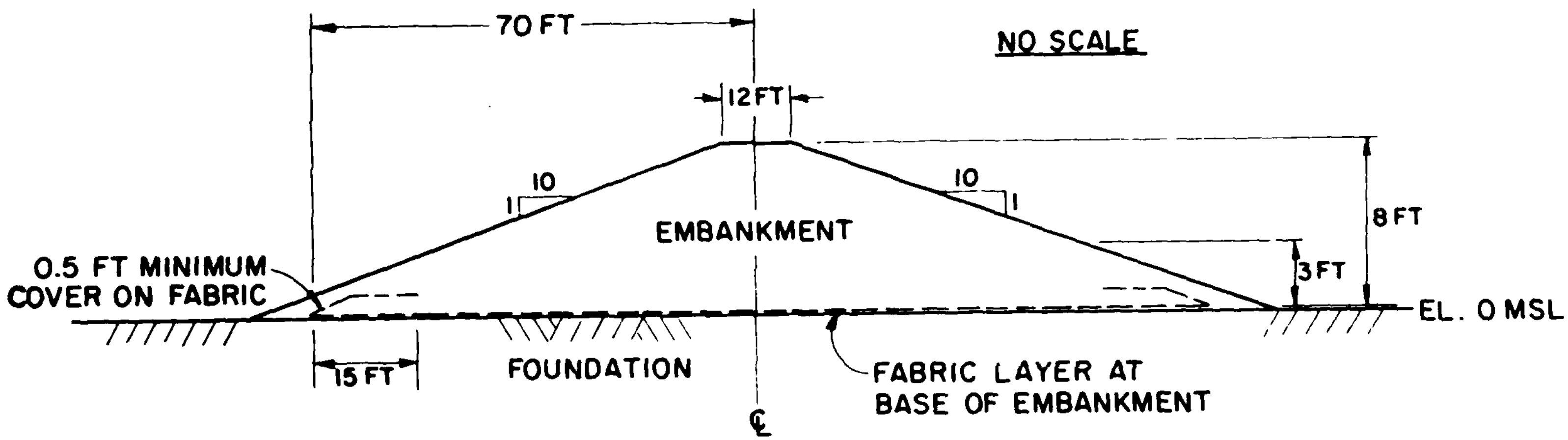



Figure 17. Simplified Fabric-Reinforced Embankment Section Used for Analysis Purposes

consolidated soils, a 40-ft sediment thickness, an average initial void ratio of 2.7, and compression index $C_c = 0.8$. Based on these values, settlement of about 3 ft was computed for the initial dike height to el 8.

2. Embankment construction material consisted of a fine, poorly graded, semi-angular, fairly clean Mobile sand (SP) with 100 percent passing the U. S. No. 10 sieve, 83 percent passing the U. S. No. 40 sieve, and 2 percent passing the U. S. No. 100 sieve, with a uniformity coefficient of 1.3. These data were determined on material taken from nearby dredged material containment areas.

3. It was assumed that the sand would be placed in a loose condition in the embankment (tests conducted on the sand alone and sand-fabric indicated that the friction angle, $\phi_{sf} = 30^{\circ}$, was essentially the same).

4. The unit weight of the sand embankment material was estimated to be 100 pcf above the permanent water table and 60 pcf below the water table.

5. Field vane shear tests and laboratory tests conducted by the WES indicated that the unconsolidated undrained shear strength of the foundation materials prior to construction were cohesion c = 50 psf from the surface to a depth of el -5; c = 100 psf from el -5 to -15; and c = 150 psf from el -15 to about el -40 where a medium-dense, clean white sand was encountered. In addition to the above tests, consolidation undrained R shear strength tests conducted to predict available foundation strength for future dike raising indicated cohesion c of about 0.15 tsf and friction angle $\phi$ of about $11^{\circ}$.

These data were then used to analyze the four unsatisfactory modes of potential behavior previously described.[14]

Horizontal Sliding/Lateral Spreading of Embankment

This potential unsatisfactory behavior (Figure 14a) was likely to cause a portion of the dike to slide horizontally outward if the frictional resistance between the embankment and fabric were less than the lateral earth pressure. Another possibility was that, although the soil-fabric frictional resistance might be sufficiently greater than the lateral earth pressure to cause sliding, the tensile strength of the fabric might be insufficient, resulting in fabric failure with subsequent outward sliding of embankment and fabric along the soft foundation.

The horizontal force that might cause lateral sliding was approximated by Mohr-Coulomb active pressure.

$$P_a = 0.5\ \gamma\ H^2\ \tan^2\ (45^\circ - \phi/2)$$

or

$$P_a = 0.5 \times 100\ \text{pcf} \times (8\ \text{ft})^2 \times \tan^2\ (45^\circ - 30^\circ/2)$$

$$P_a = 1{,}070\ \text{lb/ft-width}$$

while the sliding resistance was approximated by

$$P_r = \frac{HL}{2}\ \gamma\ \tan\ \phi$$

$$P_r = \frac{8\ \text{ft} + 0.5\ \text{ft}}{2} \times 70\ \text{ft} \times 100\ \text{pcf} \times \tan\ 30^\circ$$

$$P_r = 17{,}200\ \text{lb/ft-width}$$

and the factor of safety against sliding was defined as the ratio ($P_r$/$P_a$), assuming the fabric tensile strength is not exceeded.

By inspection, the controlling parameter was fabric tensile resistance to the active pressure. A minimum factor of safety of 2.0 was chosen against sliding, which gave a required fabric ultimate tensile strength of 2 × 1070 lb/ft-width or 2140 lb/ft-width. The fabric used should meet or exceed 180 lb/in.-width ultimate tensile strength.

## Localized Foundation Bearing Failure and Rotational Subsidence of Embankment

This potential unsatisfactory behavior was analyzed by a procedure based on Modified Bishop slope stability analysis for estimating the fabric ultimate tensile strength needed to provide a factor of safety against rotational slope failure (Figure 14b) of a sand embankment on a soft cohesive foundation. The following assumptions were considered in the analysis:

1. Full fabric tensile strength is developed before slope failure.

2. Consideration of shear strength in the embankment may be neglected as tensile cracks are likely to occur.

3. The critical slip circle passes through the embankment behind the crest, is tangent to the assumed foundation strength change layer at el -5 (from c = 50 psf to c = 100 psf), and surfaces beyond the embankment toe.

4. The embankment and fabric are placed on the foundation simultaneously.

5. Foundation cohesion and ultimate fabric tensile strength are mobilized simultaneously.

6. The likelihood of internal embankment slope failure is minimal because the factor of safety against failure was $F = \tan 30^{o}/\tan 5.7^{o} = 5.8$ (where $30^{o} = \emptyset$ and $5.7^{o}$ is embankment slope).

Using the above assumed conditions, the minimum factor of safety was less than unity for no fabric and above one only if fabric was used. The required fabric strengths determined for various factors of safety are shown in Table V.

TABLE V

FACTOR OF SAFETY AND FABRIC ULTIMATE TENSILE STRENGTH

| Worst-Case Minimum Factor of Safety | Required Fabric Ultimate Tensile Strength (lb/in.-width) |
|---|---|
| 1.0 | 170 |
| 1.1 | 225 |
| 1.2 | 285 |
| 1.3 | 341 |

It was recommended that a minimum factor of safety between 1.1 and 1.2 be used and that a fabric strength giving this factor of safety be used to prevent rotational subsidence of the embankment. Therefore, a fabric with an ultimate tensile strength between 225-285 lb/in.-width was recommended for design purposes.

## Estimation of Fabric Tensile Stresses Developed from Embankment Deformation

Lack of knowledge in estimating the tensile stresses developed in the fabrics by embankment deformation (Figure 14c) presented the greatest problem of all the factors concerning analysis and design of a fabric-reinforced embankment. For design purposes it was postulated by Haliburton, Douglas, and Fowler that, once foundation bearing capacity was exceeded by embankment bearing pressure, foundation bearing failure and resulting foundation deformation would occur, thus allowing the embankment to settle.[15] The construction procedure outlined earlier should allow the fabric to be placed, anchored, and covered by embankment material before excessive embankment settlement occurred. Details for this construction sequence were shown in Figure 13.

Embankment foundation bearing failure should occur when the embankment height exceeds el 3 and deformation should occur in the embankment and fabric. It was assumed that the embankment sand would attempt to slip laterally and the fabric should carry these stresses at relatively small strains. If this movement were small, then internal arching of the sand should reduce and redistribute the effective vertical stress to outer portions of the embankment. Assuming relatively small fabric strains were allowed, the embankment would remain in one stable mass until sufficient foundation consolidation had occurred to support the embankment weight without general bearing failure. Even though initial soil shear strengths were extremely low, rapid increases in the strengths were expected to occur because the soft cohesive material contained numerous silt and sand lenses and stringers that are typically found in

such alluvial deposits. The permeable fabric and sand embankment would also allow dissipation of excessive pore pressure in the critical zone nearest the fabric.

A summary of bearing pressure and related data for the fabric embankment and for design crest elevations ranging from el 8 through four consecutive dike increments to el 25 are shown in Table VI. Estimated maximum bearing pressures were determined for the fabric located at el 0 and the minimum foundation capacity data were obtained by extrapolating results from the unconsolidated undrained (R) strengths. It may be noted from Table VI that only the initial construction conditions to el 8 indicate that embankment bearing pressure exceeds foundation bearing capacity. Initial factors of safety without fabric were about 0.4 at el 8, but for subsequent raises of the unreinforced embankment, they varied from 1.5 to 1.8. These values were assumed to be so-called "worst case" because no foundation consolidation was considered; therefore, it was assumed that if the fabric-reinforced embankment could be initially constructed without failure then subsequent raises of the dike could be achieved after excess pore pressure dissipation.

Even though deformation would continue to occur in the center portion of the dike, the frictional force caused by internal embankment incipient sliding would have to be carried by the fabric. These frictional forces were calculated for subsequent dike raises by the product of embankment weight and the tangent of the angle of internal friction, $\phi_{sf}$, between the soil and fabric and are tabulated in Table VI. It was concluded that the maximum friction force at el 8, using a $\phi_{sf} = 30^{o}$, would yield a maximum tensile force of 460 lb/ft-width or 38 lb/in.-width in the fabric. It was also concluded that this frictional

TABLE VI

BEARING PRESSURES AND RELATED DATA FOR EMBANKMENT[5]

| Design Crest Elevation (ft MSL) | Expected Maximum Consolidation from Previous Load (ft) | Estimated Maximum Bearing Pressure BP (psf)[1] | Minimum Foundation R Shear Strength (psf) | Minimum Foundation Bearing Capacity BC (psf) | BC - BP (psf) | Bearing Factor of Safety w/o Fabric BC/BP | Minimum Horizontal Force at Soil-Fabric Interface[2] (lb/ft-width) | (lb/in.-width) |
|---|---|---|---|---|---|---|---|---|
| 8 | 0.0 | 800 | 50 | 290 | -510 | 0.4 | 460 | 38 |
| 12 | 2.9 | 1,380 | 400 | 2,280 | 900 | 1.7 | 800 | 67 |
| 16 | 4.1 | 1,850 | 570 | 3,240 | 1,390 | 1.8 | 1,070 | 89 |
| 20 | 5.0 | 2,300 | 660 | 3,760 | 1,460 | 1.6 | 1,330 | 111 |
| 25 | 5.7 | 2,860 | 750 | 4,260 | 1,400 | 1.5 | 1,650 | 138 |

[1] $\gamma_m$ assumed 100 pcf about W.T., $\gamma'$ assumed 60 pcf below W.T., W.T. at el 0 MSL.

[2] Computed as (BP tan $\phi_{sf}$) × (1 ft-length), $\phi_{sf} = 30^\circ$.

force would be the most critical case since the initial assumption of frictional force caused by the difference in pressure between the bearing pressure and bearing capacity for subsequent dike raises did not consider the forces caused by consolidation settlement that would result in repeated embankment settlement/incipient sliding/arching behavior after each dike increment.

Assuming that the most critical case would occur and applying a factor of safety of 2.5 to 38 lb/in.-width, fabric strength of at least 95 lb/in.-width or about 100 lb/in.-width was needed to provide satisfactory embankment reinforcement. It was also assumed that the selected fabric should not develop more than 10 percent fabric elongation at 100 lb/in.-width, which assumed that about four percent strain would occur to carry the stress from the maximum bearing pressure of the embankment. The ultimate fabric strength in tension, at el 25 (Table VI), was 138 lb/in.-width or about 140 lb/in.-width and was necessary to carry the maximum horizontal forces.

Estimation of Fabric Pullout Resistance

It was postulated earlier in this report that the center portion of the embankment would subside and cause fabric tension and that the embankment bearing pressure on the outside portion near the embankment toe would be less than the bearing capacity; therefore, this section would be in a more or less stable condition, acting as weight to anchor the ends of the fabric to prevent pullout due to tensile stresses. The maximum horizontal stress in the fabrics, shown in Table VI, was estimated to be about 460 lb/ft-width for el 8. Using the section shown in Figures 14d and 17 (note 15-ft overlap), but assuming the fabrics were placed at el 1, the minimum

anchorage force for this condition was expected to be 15 ft ×

$\frac{(2\text{ ft} + 0.5\text{ ft})}{2} \times 100\text{ pcf} \times \tan 30^{\circ} \times 2\text{ sides} = 2170\text{ lb/ft-width};$

therefore, the factor of safety was estimated as 2170/460 = 4.7, which did not consider the effects of overlapping. Thus, based on the above computations it was concluded that fabric pullout was highly unlikely under the estimated working stresses of the fabric.

## Fabrics Selection and Placement in the Test Section

Based on the fabric strengths determined in the foregoing discussion, the following fabric conditions were required:

1. To prevent horizontal sliding of the embankment: 180 lb/in.-width ultimate tensile strength.

2. To prevent rotational subsidence of the embankment: between 225 and 285 lb/in.-width ultimate tensile strength.

3. To support anticipated embankment deformation under working loads: 100 lb/in.-width at 10 percent elongation and 140 lb/in.-width ultimate tensile strength.

The fabrics selected that met or exceeded the above requirements tested by Haliburton, Anglin, and Lawmaster were identified as Nicolon 66475, Nicolon 66186, Advance Type I, and Polyfilter X.[16] A summary of the laboratory data obtained from tests conducted on these four fabrics is contained in Table IV. All four fabrics were recommended for use in the test section, on grounds that the experimental nature of the project justified evaluation of the greatest number of potentially applicable materials currently available on the market and that data from this test section would allow cost-effective fabric selection for the remaining portion of the Pinto Island embankment and other future construction.

Suggested placement of the fabric in the test section was that Advance Type I fabric be used as reinforcement from sta 0+00 to 2+00, Polyfilter X fabric from sta 2+00 to 4+00, Nicolon 66475 fabric from sta 4+00 to 6+30, and Nicolon 66186 fabric from sta 6+30 to 8+30 (see Figure 10). The fabrics were to be placed transverse to the longitudinal axis of the embankment in 18-ft widths for the Advance Type I and Polyfilter X and 5 m (16.4 ft) widths for the Nicolon fabrics. Advance Type I and Polyfilter X are woven in 6-ft widths that are then factory sewn to 18-ft widths. The fabrics were to be overlapped and sewn with a portable field sewing machine capable of chain stitch sewing with polyester thread. The construction sequence was described earlier in this chapter.

## Instrumentation Requirements

Instrumentation of the test section was essential to determine whether proposed construction sequences and fabric placement techniques would provide the desired results, that actual dike and foundation behavior agreed with predicted behavior, and to provide data to determine when future incremental raising should take place, both during and after initial construction of the embankment. Required information included the relative horizontal and vertical movements of the embankment, especially during construction, and the excess pore pressures generated in the foundation, both during and after construction. It was recommended that the instrumentation be limited to those types that were simple, work properly under all field conditions, and had a proven history of effective performance. It was therefore recommended that the

following instruments be installed at every 100-ft station along the center line of the dike:

1. Casagrande-type porous stone piezometers were to be placed at the outside edges and center of the dike in clusters of four at depths of 5, 10, 20, and 30 ft below the surface by the Foundation and Materials Branch, MDO.

2. Horizontal and vertical settlement plates were to be placed by the Foundations and Materials Branch, MDO, at five locations along the transverse axis of each 100-ft station. The settlement plates were to consist of 18-in. square plates, 3/4-in. thick, with 3/4-in. steel pipe risers to accommodate a survey target. The plates were to be installed directly on the fabric at the center line, at each toe of the embankment, and at each mid-point between the toe and center line.

Initially, temporary control points, far enough from the dike boundaries to prevent disturbance during construction, were to be installed and permanent control monuments were to be installed as soon after completion of construction as possible. Piezometers and settlement plates were to be installed as soon as possible.

All piezometers were to be read and plotted every 24 hr to avoid any dangerous pore pressure problems during construction. Horizontal and vertical control data were to be collected daily and plotted to detect any potential trends of excessive settlement and/or lateral movement during construction. Once construction was completed, frequency of readings could be decreased to weekly, monthly, or whenever necessary.

It was agreed that a qualified geotechnical engineer should be responsible for installation of instrumentation and evaluation of the

data collected to determine if the field conditions were in reasonable agreement with those assumed for design purposes and to make any necessary changes in construction procedures.

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DESIGN, CONSTRUCTION, AND ANALYSIS OF FABRIC-REINFORCED EMBANKM--ETC(U)
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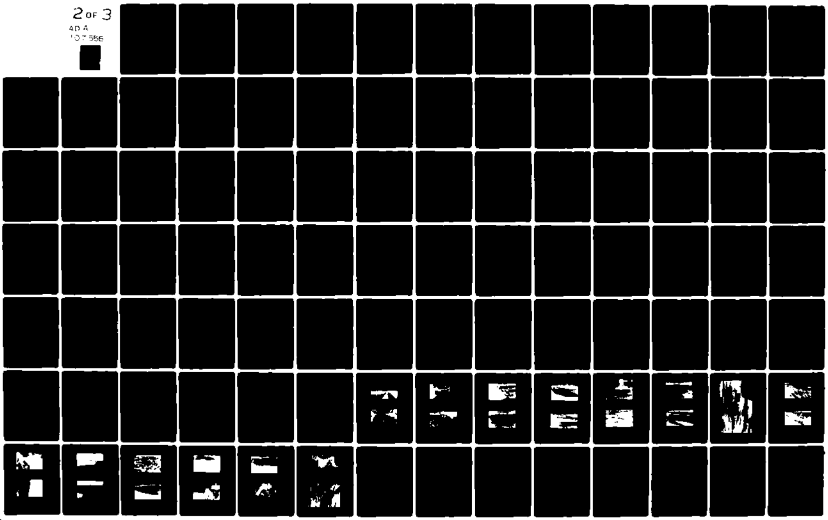

## ENDNOTES

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[9]Haliburton, Douglas, and Fowler, pp. A4-A8.

[10]T. Allan Haliburton, Cyd C. Anglin, and Jack D. Lawmaster, "Selection of Geotechnical Fabrics for Embankment Reinforcement" (School of

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[11] Haliburton Associates, p. 27.

[12] Haliburton Associates, pp. 9-26.

[13] Haliburton, Douglas, and Fowler, pp. A18-A19.

[14] Haliburton Associates, pp. 17-19.

[15] Haliburton, Douglas, and Fowler, pp. A2-A3.

[16] Haliburton, Anglin, and Lawmaster, pp. 51-52.

# CHAPTER V

# CONSTRUCTION AND ASSESSMENT OF RESULTS

## Introduction

Construction of the fabric-reinforced embankment test section was begun on 26 October 1978 and completed 11 January 1979. The purposes of this Chapter are to present the test results, assess the construction procedure (photographic construction sequence, Appendix B) and performance of the test section, and analyze and compare the field data (Appendices A and C) with theoretical design charts (Appendix D) developed to predict the proper fabric tensile strength for an embankment constructed on very soft foundation materials.

## Borrow Sites

Three potential borrow sites had been selected prior to construction, but easement problems precluded their use. The three sites actually used contained similar fine to medium sands, interlayered with variable amounts of fine-grained silts and clays in substantially thinner layers. The thinness of the borrow layer required considerable movement of the dragline, and continual construction of new haul roads was necessary to maintain a continuous supply of material for dike construction. In future dike construction activity at Pinto Pass, every effort should be made to obtain borrow removal rights from originally selected sites because the material contained in these areas is a high quality sand with minimal fine-grained soil layers near the surface.

## Access and Haul Road Construction

Access and haul roads constructed by the contractor performed satisfactorily during conduct of this work. The main access road leading into the site and the north abutment of the test section and equipment storage and parking area were reinforced with waste ALCOA fabric and covered with about 2 in. sand and 3 to 4 in. reef shell furnished by the Government. ALCOA fabric was a very strong woven polypropylene fabric discarded after use as a filter in a bauxite ore processing plant. Compaction was accomplished by rolling with dump trucks after wetting down with a water truck. Approximately 1000 ft of the main access road (with one culvert) was constructed through a thickly wooded area that was cleared prior to road construction.

Most of the roads throughout the borrow site, shown in Figure 9, Chapter IV, consisted of existing red clay-sand that underwent considerable rutting from loaded truck traffic and required periodic maintenance by one of the two small, wide-track dozers and the water truck. During the early part of the contract, the water was applied to keep the primarily cohesionless soil in the roadways wet, but later, during cooler and wetter weather, enough moisture was maintained on the haul road surfaces through natural action to maintain reasonable amounts of apparent cohesion. It should be noted that haul roads reinforced with the waste ALCOA fabric required the least amount of maintenance. Borrow material hauled to the site was sufficiently moist not to require application of water on the test section.

## Equipment Rental Contract and Borrow Operation

Equipment rental contracts are commonly let by CE Districts so that

the Districts may maintain direct control over the performance of contractors, equipment, and personnel. Estimates of the time, equipment, necessary labor, and contractor- or Government-furnished materials required to complete this project were made by Haliburton Associates.[1] A list of items needed for the rental contract is shown in Table VII.

The 50-ft boom dragline used on the job did not require use of wooden mats to maintain mobility since the borrow area was primarily sand and provided adequate support. On an average, the dragline was able to fill a 10-cu-yd truck in about seven drags. To meet the requirement of the contract and to reduce the number of drags required per load, the drag bucket sides had been extended to increase its capacity to 1-3/4 cu yd. However, the extensions separated from the bucket, and the trucks quite often hauled less-than-capacity loads. Because of the number and distance of swings necessary to locate and selectively borrow quality cohesionless material, it became impossible to achieve the maximum 150-cu-yd-per-hour borrow rate of the dragline. Consequently, it is recommended in future operations that a dragline with a 2-1/2- to 3-cu-yd capacity, with or without mats, be specified; a better borrow area with cleaner sand would result in less equipment movement and a more efficient borrow and haul operation.

The loads hauled by the 10-cu-yd (struck capacity) tandem-axle dump trucks, which weighed 17,000 lb unloaded and 47,000 lb fully loaded, were kept lower than capacity because of the poor support provided by the soil beneath the test section and the unknown factors concerning the support capabilities of the soil-fabric reinforcement. As construction progressed, the trucks were able to operate satisfactorily with a minimum amount of road maintenance in the borrow area and along the

TABLE VII

ITEMS NEEDED FOR RENTAL CONTRACT CONSTRUCTION[1]

| Bid Item No. | Quantity | Description | Total Hours Authorized for Quantity |
|---|---|---|---|
| 1 | 2 | Small wide-track dozer with blade and operator, IH HD500 or equivalent, maximum ground pressure 2.5 psi | 520 |
| 2 | 3-6* | Dump truck and operator, 10-cu-yd struck capacity, short wheelbase, tandem axle (larger trucks not acceptable). | 940 |
| 3 | 1 | Water truck with pump or water supply and operator, 1,500- to 2,000-gal minimum capacity, self-filling, minimum 8-ft-wide rear spray bar. | 260 |
| 4 | 1 | Dragline and operator, 1-3/4-cu-yd struck bucket capacity (welded sideboards acceptable), 50-ft minimum boom length, furnished with mats sufficient to lower average ground pressure to 2 psi. | 210 |
| 5 | 2 | Portable (field) sewing machine, power source, and operator, Fischbien Model D or equivalent single-needle type capable of field-sewing lapped seams of civil engineering fabric (filter cloth) with No. 43-No. 53 cord multifilament polyester thread. Thread to be supplied with machine. | 120 |
| 6 | 2-4* | Laborer, common | 600 |

*Quantity of this item will vary depending upon particular phase of the work. The Government will give 24-hr notice when a change (addition or deletion) of the number of items in use is comtemplated.

outer edges of the dike where double fabric reinforcement was provided near the dike toe. At least 2-1/2 to 3 ft of sand fill material was required on top of the fabric before the two parallel roadways on either side of the dike were deemed firm enough to support loaded trucks and allow them to be backed into the work area. High pore water pressure and dump truck activity caused some liquefaction of the sand fill resulting in occasional miring of trucks, but this was generally overcome by selectively dumping and spreading the fill toward the outside and center of the dike. Pore pressure in sand boils and liquefied areas that occurred immediately after the spreading operation were usually dissipated sufficiently after 24 hr to allow support of loaded trucks.

Based on the observed capability of the loaded 10-cu-yd dump trucks to negotiate haul roads and dike sections, it is suggested that future construction incorporate the use of larger capacity dump trucks (12 to 15 cu yd). Use of larger trucks should improve the efficiency of the borrow operation and prevent the bottlenecking in the fill area that was a frequent problem in this project.

To summarize, borrow operations proceeded relatively well, but not at a rate that was deemed most efficient or desirable. The haul area was approximately one mile long and relatively flat and rolling resistance was minimal. Initially, it had been estimated that the dragline/dump truck operation should yield at least 100 cu yd per hr, but this volume was achieved only about 17 percent of the time. During construction, the actual average rate was about 80 cu yd per hr. This low efficiency was the result of a number of factors, such as the poorer quality of the borrow material available compared to that of the previously selected sites; the poor condition of the contractor's equipment,

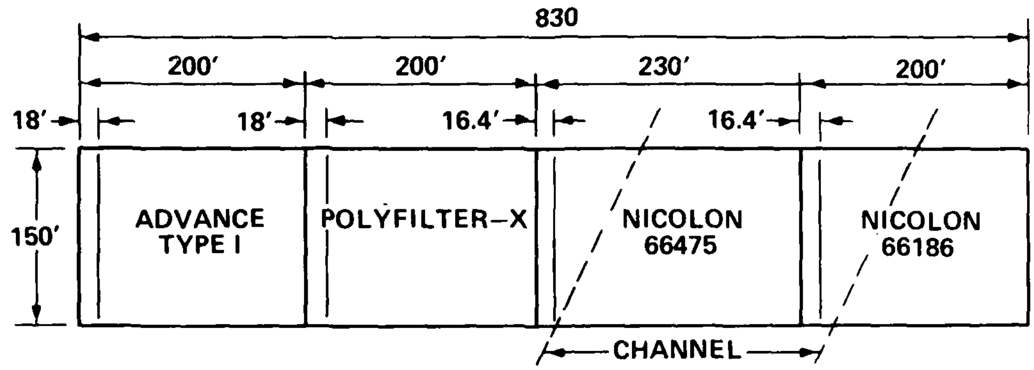
which was subject to frequent breakdown and constantly in need of repair and maintenance; and the lack of driving skill exhibited by about half of the dump truck drivers. Slowdowns resulting from stuck trucks were more often the result of driver error than of road conditions. Use of larger, well-maintained equipment and more skilled operators in future dike construction should appreciably improve the efficiency of borrow removal activities.

## Installation of Fabric

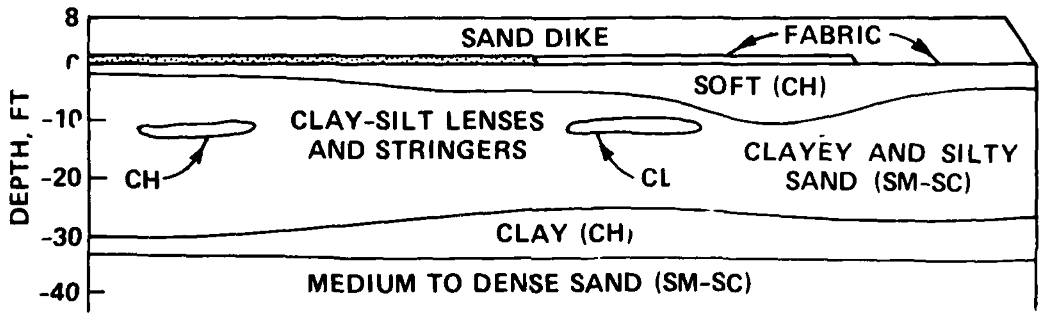
### Advance Type I

Prior to placement of the Advance Type I fabric, a sand layer was spread to act as a smooth working table on which to place and position the fabric for sewing and eventual coverage. Initial construction consisted of spreading approximately 1 ft of sand over a grass mat from sta 0+05 to about sta 1+98 without too much difficulty. A plan view of the fabric layout for each section and a soil profile are shown in Figure 18. The dragline operator was instructed to selectively borrow only dry sand during construction of the working table because the wet sand had a higher unit weight and the extra weight was causing excess pore water pressure, resulting in liquefaction and loss of support for the lightweight wide-track dozers.

It was observed that, within 24 to 48 hr after sand layer placement, pore water pressure would exceed the fill height and water would puddle or run off. To prevent rutting and miring of equipment, it was determined that the fabric should be laid on the working table as soon as a segment of the table, long enough to accommodate several widths of fabric, had been completed. The 200-ft lengths of the 18-ft-wide (three 6-ft widths, factory sewn) fabric were positioned on the sand layer




A. PLAN VIEW OF EMBANKMENT TEST SECTION




B. TYPICAL SOIL PROFILE, PINTO PASS, ALABAMA

Figure 18. Plan and Profile, Embankment Test Section

transverse to the longitudinal axis of the dike and sewn together with a hand-held sewing machine similar to those used to close animal feed sacks.

One problem encountered with the Advance Type I fabric was the presence of transverse seams where mill ends had been joined. This seaming constituted a potential failure area in the fabric since tensile stresses would be developed across the seam from the unbalanced transverse forces generated by the embankment. This problem could have been avoided by specifying continuous fabric lengths in the purchase contract. It was also noted that some of the factory seams were incompletely joined where the sewing had been done too close to the edge, catching only one of the two pieces of fabric. However, these problems did not have any effect on the placement or construction procedure or the embankment performance, but they could be a potential problem where the embankment or construction procedure might rely heavily on the strength of factory-sewn seams.

There were no other particular problems with field use of the Advance Type I fabric, and the contractor's personnel experienced no problems sewing this relatively thin woven fabric together with the Fischbien sewing machine. Each lap was sewn together with three rows of stitching, and the loose end of the thread was tied back through a loop of the chain stitch to prevent unraveling at the end of each seam or when the thread broke or a spool was finished. The Fischbien was capable of operating on 110-volt AC or 24-volt DC and providing a single thread chain stitch. A portable generator was isolated from wet ground conditions by placing it on pieces of fabric used to protect the fabric rolls during shipping. A heavy-duty electrical three-conductor drop line was

provided by the contractor. The contractor's personnel learned to operate the sewing machine without too many problems other than occasionally breaking the thread or needle and the minor thread-tension adjustments and cleaning and oiling that were necessary each day.

Even though the sand working table was reasonably flat and working conditions were favorable, there was always a minor degree of wrinkling at the seams. This condition may have been the result of variations in sewing thread tension or the greater resistance to stretching that can be noted at the selvage edge of any fabric. These wrinkles, however, were minimized by the construction scheme used in constructing two parallel access roadways near the outside edges of the embankment prior to covering the center portion of the fabric. Continued maintenance and monitoring of this procedure was necessary for proper employment.

Once the fabric at each toe was covered with approximately 1 ft of sand fill material, the outside edge of the fabric at the foldback point was back-dragged with the dozer blade and finished by hand labor with shovels to provide a straight edge for the foldback. Details of this construction procedure are shown in Figure 19. This procedure was time-consuming and caused a bottleneck in construction operations. Once the fabric was folded back, it was covered with about 1 ft of sand fill material and truck traffic was then allowed to back out onto the double-reinforced roadway. The width of foldback fabric varied from 30 to 35 ft instead of the previous 15 ft called for in the design because the working table elevation was higher than expected. The anchoring benefit derived from this technique for an embankment with a wide base and moderate slope like the Pinto Pass dike section is questionable and should be investigated for actual effectiveness. Location of the outer edges of the haul road was limited by the foldback edge of the fabric and location of the settlement

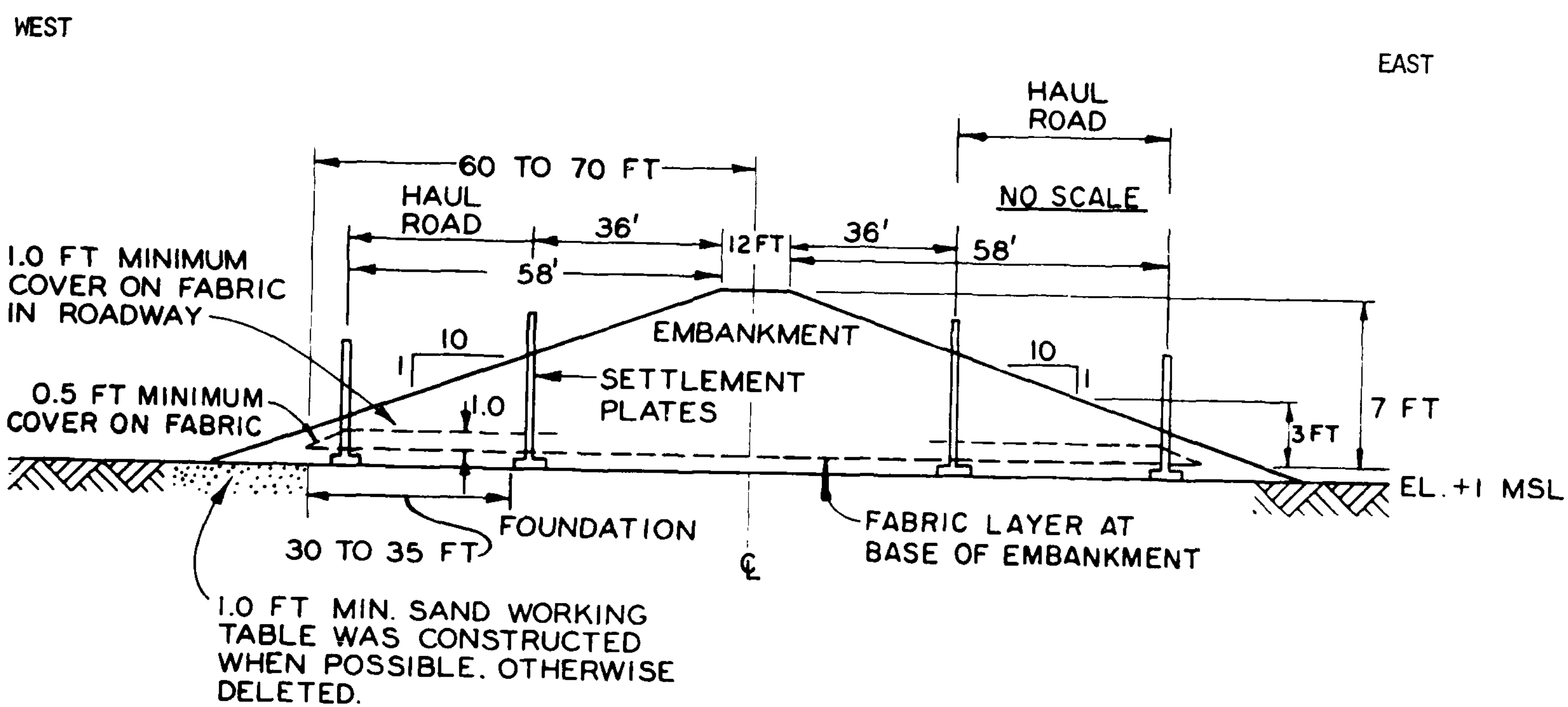



Figure 19. Construction Details of Actual Fabric-Reinforced Embankment Section

plate resting on the fabric before folding back into the roadway. One direct benefit was the supporting capability that the double fabric afforded the heavy truck traffic. Rutting was minimal, and as long as the trucks stayed on the double folded back fabric roadway, they experienced no problems. Trucks that were inadvertently driven off the roadway became mired to their beds and had to be unloaded in place and pushed and pulled back to firm ground by the two dozers.

## Polyfilter X

The Polyfilter X fabric was installed from sta 1+98 to 4+00 using essentially the same placement technique as had been used with the Advance Type I fabric. Working conditions, however, grew worse as the operation advanced onto progressively softer foundation materials. Surface features consisted primarily of reeds and cattails without a grass mat or well-developed crust. Also, a portion of this segment was in the tidal zone. Due to the rapid rise of pore water pressure, advancement of the working table, prior to fabric placement, was restricted to increments of 100 ft or less. It was also difficult to maintain the 1-ft thickness of the working table. Dozer operators, from fear of losing the dozers and desire to maintain equipment mobility, tended to increase the thickness of the working table to 1.5 or 2 ft. The more passes that were made with the dozers in spreading the fill, the greater the likelihood that mud waves from the underlying remolded plastic clay would break through the fill and engulf the wide-track dozers. By restricting the number of dozer passes to three or less, it was eventually possible to achieve a fairly consistent 1-ft thickness in the working table.

The increasing softness of the foundation materials created another problem in fabric placement and joining. As the working table was advanced, the dump trucks were backed on to the dike over the in-place fabric to the leading edge of the fabric where the fill material was unloaded to be spread by the dozer. This repeated traffic created large displacements at the leading edge of the fabric, which interfered with proper tensioning of the fabric and considerably slowed the entire operation, as it was necessary to dig out the leading edge of the fabric before another fabric strip could be positioned and sewn. The mud waves and quick conditions of the working table also hindered hand-labor placement of the fabric.

The Polyfilter X fabric, like the Advance Type I, was provided by the manufacturer in 18-ft widths (three 6-ft factory-sewn widths), which were field sewn with the Fischbien machine. The ends of the fabric were lapped as before, and the two parallel roadways along each toe were extended with the embankment. Despite the softer underlying soil, these haul roads sustained the heavy truck traffic with only minor rutting and required only minimal dozer maintenance.

Nicolon 66475

The use of Polyfilter X was discontinued at about sta 4+00 near the northern edge of the Pinto Pass channel, and extension of the embankment was continued with Nicolon 66475 fabric. This fabric was provided in continuous widths of 5 m (16.4 ft) and was considerably heavier than the two previously used fabrics. At sta 4+00 it became extremely difficult to construct the working table. As the embankment neared the channel, grading and spreading the working table fill became exceedingly difficult

and progressed at so slow a pace that the whole project was virtually brought to a standstill. The unconsolidated surface channel material was extremely soft and would not support the working table and dozer spreading the material without creating a displacement section. Material near the surface in the channel at el 0.5 was near the liquid limit and had never had an opportunity for grass growth or crust formation because of tidal activity.

Only two widths of the Nicolon 66475 fabric had been installed, advancing the embankment to about sta 4+30, when it was determined that it would be impossible to develop a stable working table using the previous technique and that a new approach would have to be devised to carry the embankment across the channel. The channel was approximately 230 ft wide and water depth varied from 0.5 to 2.0 ft, depending on the tide.

Since all attempts to advance the working table had failed, it was decided that an experiment should be conducted to see if it would be possible to advance the fabric without a prepared working table. Consequently, two extra rolls of Polyfilter X were unrolled, sewn together along their lengths, and rerolled to create a 32-ft-wide by 200-ft-long roll. The end of this roll was secured under the two previously placed strips of Nicolon 66475, in line with the haul road on the west toe of the test section. Proceeding across the channel, the fabric was gradually unrolled parallel to the longitudinal axis of the embankment and was uniformly covered with approximately 18 to 24 in. of fill material. It was noted that a mud wave about 1 to 2 ft high developed under and beyond the unrolled Polyfilter X fabric. This mud wave lifted the rolled fabric above the tidal water, advanced it forward, and

stretched it in a longitudinal direction. The fabric on the mud wave afforded adequate support, in a relatively dry condition, for the labor necessary to continue unrolling the fabric across the channel.

Since this technique appeared to be progressing satisfactorily on a small scale, it was decided to apply the method to incremental embankment construction.

To achieve mud wave assistance while maintaining the transverse orientation of the fabric lengths to the longitudinal axis of the embankment, each new strip of Nicolon 66475 was unrolled on top of the previously laid strip and sewn at the leading edge. The new strip was then folded out onto the advancing mud wave. The procedure of constructing the parallel haul roads on either side of the embankment prior to covering the center portion of the embankment with fill material was continued. This technique not only provided optimal access to the work area, but also provided lateral containment of the mud wave and promoted its longitudinal advancement along the center line of the dike until it subsided against the south side of the channel.

Placement of sand in the center portion of the positioned fabric, to within 5 or 6 ft of the leading edge, produced forward movement of the underlying mud wave which raised the leading edge to about el 1.5 to 2.0. The wrinkles caused by foot traffic during placement and sewing disappeared as the mud wave advanced and stretched the fabric. This construction technique proved to be very effective in that there was no excessive build up of mud along the center line of the dike and it created an excellent working table, well above tide levels.

Walking or jumping up and down on the fabric after it was placed on the mud wave was very much like walking on a giant waterbed. If one

accidentally stepped off the fabric edge, he would sink to his waist in mud. The Nicolon 66475 fabric had more of a mat stiffness and was easier to walk on prior to placement of sand cover than either the Advance Type I or the Polyfilter X fabrics. Each roll of Nicolon 66475 fabric weighed over 500 lb, required a dozer to tow the roll from the stockpile area to near the placement area, and required about four to five laborers to unroll and stretch the fabric into position for sewing.

Placement of the Polyfilter X fabric across the channel was continued along the edge of the dike prior to placement of the Nicolon 66475 fabric, to provide passage for the survey personnel to the opposite side of the Pinto Pass channel. Even though this fabric was laid longitudinally or parallel with the alignment, it provided a localized increase in the support capacity of the Nicolon 66475 fabric, evidenced by the fact that displacement on the east side of the embankment, without the Polyfilter X reinforcement, was about twice as great as the displacement on the west side. Longitudinal placement of a strip of inexpensive fabric prior to placing heavier fabric strips transverse with the alignment may prove beneficial in reducing the overall embankment displacement and final elevation, and should be investigated in future applications.

As sand was placed on the fabric behind the leading edge, there was an abrupt displacement of the mud wave, such that the fabric was pulled down as shown in Figure 20 from about el 1.5 to 2.0 down to about el 0.5 to 1.0 over a rather short distance. This displacement caused relatively high tensile stresses across seams of the fabric, resulting in tensile failure in the thread used to sew adjacent fabric strips together. Additional seams were sewn to increase the strength, but gaps

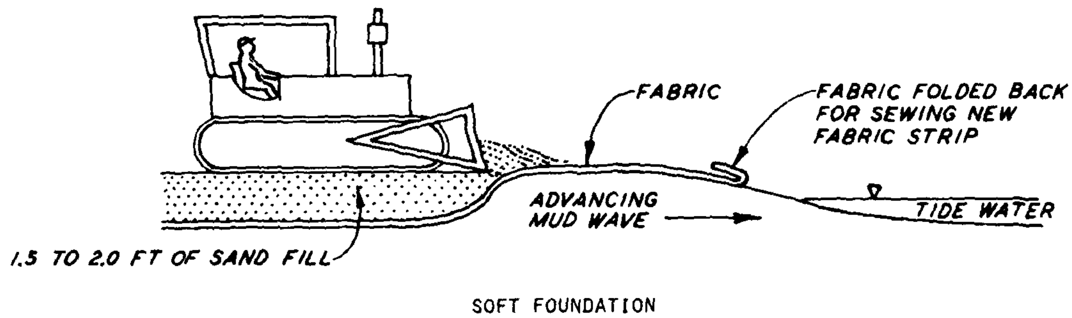



Figure 20. Laying Fabric on Mud Wave

continued to appear in the seams and additional pieces of fabric were required to cover the gaps before covering with sand. In one instance it was necessary to use a whole roll of fabric to cover a seam that developed multiple gaps.

The thread provided by the contractor proved not to be the thread specified in the contract and was replaced with 100-lb-test No. 12 nylon fishing line. The Fischbien sewing machine's needle would not accommodate the nylon line and, since needles with larger eyes were not readily available, a Sac-Up Model BB sewing machine was acquired to complete the sewing operations. Use of this high-strength thread stopped tensile failure in the fabric seams for the remainder of construction.

Although sewing was assumed to be the most appropriate method of connecting the fabric strips in this project, other techniques such as stapling and joining with loops and ropes might have proven equally successful. To minimize seam failure, additional research and testing should be done to determine the various applications of the several different methods before future projects are planned.

### Nicolon 66186

The Nicolon 66475 fabric was terminated at sta 6+38 and the last section of the embankment was constructed with Nicolon 66186 (5-m seamless widths) from sta 6+38 to 8+20 with about a 20-ft foldback at the end of the dike. This fabric was much more flexible and considerably lighter and easier to unroll and place than the Nicolon 66475 fabric, but it had a marked capacity for water absorption.

The Nicolon 66186 was placed directly on the mud wave, following the same procedure as outlined in the previous section, until the mud

wave dissipated. Thereafter it was placed directly over cattails and reeds and finally onto the grass mat area above the tidal line on the south bank of Pinto Pass.

There was more wrinkling of the Nicolon 66186 and it was more difficult to walk on than the other fabrics, but movement of the mud wave stretched and smoothed this fabric as effectively as it had the Nicolon 66475. Fabric placement directly onto the relatively level vegetated surface without prior placement of a working table was satisfactory, but required labor to walk the fabric down since its weight alone was not sufficient to flatten underlying vegetation.

During placement of the sand fill on the Nicolon 66186 fabric, a considerable amount of liquefaction and sand boils was observed as the dozers spread the fill. Water seemed to saturate and flow through the Nicolon 66186 fabric very easily, but the sand boils usually subsided within 15 min of eruption and did not pose any particular problems.

After the fabric had become wet and was being folded back into the dike section for anchoring, it appeared to become more extensible. Because it tended to curl at the edges, it was more difficult to lay flat and cover with sand. Placement of the Nicolon 66186 presented no major problems and the final section was completed at a fairly rapid rate because the working table was omitted.

## Completed Test Section

The sequential test section construction scheme outlined earlier in this report was followed as originally planned (see Figure 13, Chapter IV). After all fabric was placed, covered, lapped, and covered, the remaining fill required to complete the test section to the proper grade

was placed according to the original scheme of placing material on the outer edges first and then toward the center of the alignment until it was topped out. Placement of the fabric and fill was considered a success and there were no large subsidences, lateral spreading, tension cracks, or other adverse behavior observed along the alignment.

The contractor final-graded the embankment without difficulty and then cleaned up all the fabric wrappings and remnants, marked the location of all settlement plate riser pipes and piezometers with wooden stakes and flagging, and seeded and fertilized the embankment section without any problems. Appendix B illustrates photographically the construction sequence employed at the Pinto Pass test section.

Since the use of the mud wave as a working table was considered to be an exceptionally effective and innovative utilization of conditions that might have otherwise proved totally prohibitive to completion of the test section, the technique is recommended for construction of the 2200-ft embankment across the east end of Pinto Pass.

## Assessment of Contractor Performance and Construction Procedure

Each construction operation in the sequential construction procedure previously outlined in the text was found to be relatively critical in ensuring future satisfactory performance of the dike test section. Also, if these sequential operations are not followed and failure occurs, remedial attempts may prove futile.

The contractor's on-site personnel lacked an understanding of the engineering basis for fabric reinforcement. This factor tended to inhibit the recognition of unexpected problems and the development of

workable solutions during the initial phases of construction. Since Mobile area (and most other) contractors lack knowledge and experience in construction of fabric-reinforced embankments, it is concluded that construction of future fabric-reinforced embankments should be conducted by rental contract under the direct supervision of District engineering personnel.

Use of wide-track dozers was found to be the key to the successful completion of dike construction and any future contracts should require dozers of equal or lower ground pressure. Dozers of higher ground pressure could be detrimental to embankment construction and should not be allowed to remold and damage existing grass mat cover or crust.

The mobility and general performance of the tandem wheel 10-cu-yd dump trucks on the parallel haul roads was quite satisfactory, and it is recommended that increased efficiency might be achieved by allowing the use of 12- to 15-cu-yd capacity trucks in future contracts. Also, contracts for future operations should specify experienced truck drivers operating dump trucks that are in good condition and do not require continual repair and maintenance.

Once the contractor understood the purpose of the fabric reinforcement and recognized the uses of this new construction technique, a reasonable amount of cooperative effort to provide the necessary level of equipment and labor was realized. Except for numerous truck breakdowns and occasional dragline repairs, the construction rental equipment appeared to operate satisfactorily.

In view of the problems encountered with seam failures and improper thread, it is recommended that a Sac-Up Model BB sewing machine be used instead of the Fischbien, because the Sac-Up machine will accommodate

the heavier, larger diameter, stronger thread required to prevent seam failures. Until further testing is conducted on different sewing methods and thread sizes, it is recommended that a bonded No. 12 nylon thread of 100-lb or greater tensile strength be used in future fabric projects.

## Construction Costs

A detailed cost breakdown for the equipment rental contract, fabric, and reef shells (used for access road construction) is shown in Table VIII. Rental contract costs including sewing machine and labor costs totaled \$108,355. Fabric costs were \$43,570 and the reef shell costs were \$2,625. The total cost for the rental construction and the Government-furnished materials (fabric and shells) was \$154,455. The unit cost was therefore \$154,455/830 ft of dike or \$186/ft, and the progress made per day to build this dike was 830 ft/50 days (less delays, holidays, and Sundays) or about 17 ft/day. These costs do not include planning, design, construction, inspection, surveying, site exploration, drilling, field and laboratory soil testing, fabric testing, real estate acquisition, instrumentation and data collection, or report preparation costs.

Cost to haul 23,000 cu yd of sand fill required for dike construction, including equipment time used for access haul road construction, was determined by dividing the rental contract cost by the actual number of yards hauled, \$93,400 by 23,000 cu yd, or about \$4.06/cu yd or \$93,400/830 ft of dike or \$112.50/ft. Based on these in-place sand costs which were relatively high due to the construction technique, it is considered that if the test section had been a displacement section, construction

TABLE VIII

CONSTRUCTION COST

| | |
|---|---|
| Equipment Rental | |
| Contract Cost (Initial estimate-$110,000) | $108,355* |
| Fabric (Government Furnished): | |
| Advance Type I | |
| 43,200 $ft^2$ (4800 $yd^2$) 12 rolls 18 by 200 ft | |
| Unit Price: $0.135/$ft^2$ (minus 1% discount) | |
| or $1.22/$yd^2$ | 5,774 |
| Polyfilter X | |
| 43,200 $ft^2$ (4800 $yd^2$) 12 rolls 18 by 200 ft | |
| Unit Price: $0.145/$ft^2$ or $1.31/$yd^2$ | 6,264 |
| Nicolon 66186 | |
| 39,366 $ft^2$ (4374 $yd^2$) 12 rolls 16.4 by 200 ft | |
| Unit Price: $0.25/$ft^2$ or $2.25/$yd^2$ | 9,842 |
| Nicolon 66475 | |
| 55,773 $ft^2$ (6197 $yd^2$) 17 rolls 16.4 by 200 ft | |
| Unit Price: $0.3889/$ft^2$ or $3.49/$yd^2$ | 21,690 |
| | $ 43,570 |
| Reef shells (Government Furnished): | |
| 352 tons at $7.50/ton at supplier's stockpile | 2,625 |
| TOTAL COST | $154,455 |

* Detail of time and cost of rental contract:

| Number | Item | Time Estimate hours | Time Used hours | Item Cost per hour | Actual Cost ($) |
|---|---|---|---|---|---|
| 2 | Wide-track dozers | 520 | 695 | $45 | $ 31,275 |
| 6 | Dump trucks | 940 | 1159 | 30 | 34,770 |
| 1 | Dragline | 210 | 272.5 | 70 | 19,075 |
| 1 | Water truck | 260 | 207 | 40 | 8,280 |
| 2 | Sewing machines | 120 | 94.5 | 45 | 4,253 |
| 4 | Laborers | 600 | 713.5 | 15 | 10,702 |
| | | | | Total | $108,355 |

These hours relate to the original estimate of $89,896. The bid for these hours was $91,100. The contract was modified twice adding rental hours and money as well as extending the completion date. After both modifications the new contract price was $124,500. Actually earned $108,355.

costs could have very easily doubled or tripled if the soil beneath the entire length of the 800-ft test section had been very soft.

Equipment rental time and costs can be used to predict the approximate time and cost to construct a fabric-reinforced floating embankment across the east end of Pinto Pass. If the fabric anchor foldback section were eliminated, fabric cost and laying time would be substantially reduced; cost of labor and equipment would be decreased; and the efficiency of the haul, dumping, and spreading operations would be improved.

## Soil Foundation Exploration

Since there were little data concerning the foundation conditions beneath the proposed test section prior to construction, for design purposes it was assumed that the foundation conditions previously determined at the east end of the pass were about the same as those at the west end. Surface soils would not support a marsh buggy or drill rig prior to construction; therefore, all soil exploration was performed during and after test section construction, in conjunction with piezometer installation. Foundation conditions beneath the test section and various field and laboratory tests conducted are described in detail in Appendix A.

A profile view of the foundation soils beneath the test section, shown in Figure 18, indicates an unconsolidated soft, black, highly plastic clay layer about 3 to 12 ft thick with the deepest portion in the Pinto Pass channel. Beneath this clay layer, to a depth of about 25 to 30 ft, a layer of clayey and silty sand with clay and silt lenses and

stringers existed. Below this material was a fairly strong, highly plastic clay about 2 to 5 ft thick resting on medium to dense sand.

## Installation of Instrumentation

Instrumentation was installed during construction of the test section to provide data necessary to evaluate the actual dike and foundation behavior and compare it with the predicted behavior. These data could also be used to determine the proper time to further raise the embankment. Relative horizontal and vertical movement of the dike section and the excess pore pressure generated in the foundation, both during and after construction, are tabulated and plotted in Appendix C.

The locations of permanent survey monuments, settlement plates, and piezometers are shown in Appendix C. Unfortunately, delays in scheduling of drill crews and equipment caused considerable delay in placement of the permanent survey monuments and embankment instrumentation, especially the piezometers. Therefore, some data reflecting pore pressures during initial construction were not obtained. However, once instrumentation installation was begun, it proceeded without further delay. The porous plastic-point piezometers and steel pipe settlement plates fabricated by the MDO Core Drill Section performed satisfactorily, and it is recommended that this type of instrumentation be used in the remainder of dike construction, but with greater spacing along the alignment.

## Piezometer and Settlement Measurements

Piezometer and settlement measurements are shown plotted versus time in Appendix C.

During installation, the piezometer riser pipes at the center line and toe were truncated at el 9 and 5, respectively. As an unfortunate result, three initial pore pressure measurements were not obtained because overflow occurred before extensions could be added to two of the pipes at the center line and to one at the toe of the dike. However, since pore pressure continued to rise throughout embankment construction, it is assumed that the highest possible measurements were recorded.

The maximum pore pressure at the end of construction along the center line was el 11.2 at sta 5+00, tip el -9, in a highly plastic clay directly beneath the embankment (Table XV). The maximum pore pressure at the end of construction near the toe of the dike was el 10.5 at sta 6+00, tip el -9, also located in the plastic clay. Most of the piezometers located in the clayey silty sand layer showed fairly rapid dissipation after construction and now reflect changes in the groundwater table caused by rainfall and tidal fluctuation. Pore pressure measurements from piezometers located in the clay layer declined more slowly but have dissipated considerably, and the time-history curves, four months after construction, are relatively flat and stable.

Figure 21 shows a plot of settlement and pore pressure along the center line of the embankment. The maximum change in pore pressure, $h_w$ = 6.4 ft, occurred at sta 6+00. The effective foundation stress, as constructed and four months after construction, is determined by the following example for sta 1+00 through sta 7+00; the data are shown in

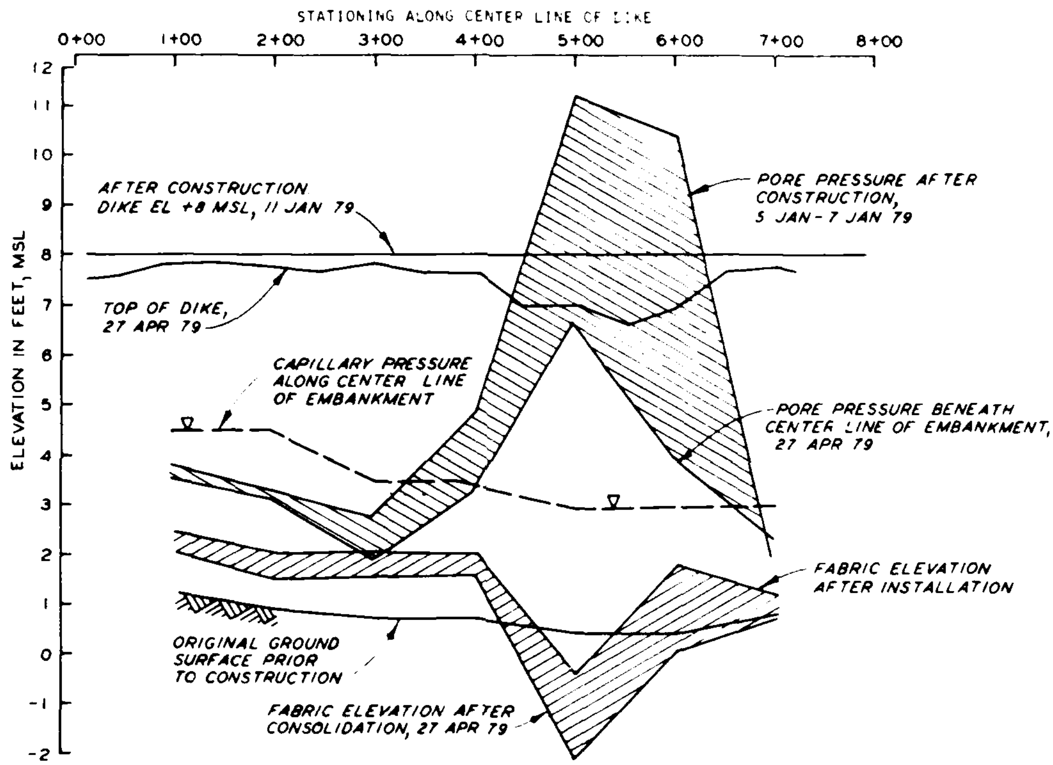



Figure 21. Settlement and Pore Pressure Along Longitudinal Center Line of Embankment

Tables IX and X, respectively.

$$\bar{\sigma} = \gamma_s H_1 - \gamma_w h_w \text{ , example for sta 5+00}$$

where $\gamma_s$ = sand fill material, 100 pcf

$h_w$ = excess pore water pressure, end of construction, 11.2 ft, minus tide elevation, 0.5 ft = 10.7 ft

$H_1$ = fill height, 8.3 ft.

$\gamma_w$ = unit weight of water, 62.5 pcf

$\gamma_w h_w$ = excess pore water, 62.5 pcf x 10.7 ft = 670 psf

$\gamma_{ws}$ = unit weight of soil when below water, 60 pcf

$\gamma_s H_1$ = total stress on fabric, $\sigma_{tc}$

$\gamma_s H_1$ = (100 pcf){8.3 ft - 0.8 ft (below tide)} + (0.8)(60 pcf) = 798 psf

$$\bar{\sigma} = \gamma_s H_1 - \gamma_w h_w$$

$$\bar{\sigma} = 798 \text{ psf} - 670 \text{ psf} = 128 \text{ psf}$$

Immediately after construction, pore pressure measurements along the toe of the dike were generally less than el 3 and declined to less than el 2.2 within four months after construction. One piezometer located at sta 6+00 (east toe, tip el -9) exhibited pore pressure at el 10.5 immediately after construction, but rapidly decreased to el 2.2 after construction. Capillary rise of the water table within the embankment varied from el 4.5, sta 1+00, on the north center line of the embankment to el 3 on the south end at sta 7+00.

## Settlement Data

Settlement plate measurements versus time in Appendix C indicate that over 90 percent of the consolidation from the imposed load occurred within four months after construction. Figure 21 shows a longitudinal center line profile view of the original ground before construction, completed dike height at el 8, subsidence of the crest since construction,

TABLE IX

EFFECTIVE FOUNDATION SOIL STRESS ALONG CENTER LINE OF EMBANKMENT AT END OF CONSTRUCTION

| Sta | *Dike Height, $H_1$ Above Fabric $H_1$, ft | Dike Height Above Foundation $H_2$, ft | Pore Pressure (el ft) | Excess Pore Pressure Minus Tide 0.5 ft ($h_w$, ft) | Excess Pore Pressure $\gamma_w h_w$ (psf) | Total Stress On Fabric $\gamma_s H_1 = \sigma_{Tf}$ (psf) | Total Stress On Foundation $\sigma_T = \gamma_s H_w$ (psf) | Effective Stress on Fabric, $\bar{\sigma} = \sigma_{Tf} - \gamma_w h_w$ (psf) | **Ultimate Bearing On Foundation $\sigma_T - 5.7c$ (psf) | ****Maximum Horizontal Force at Soil-Fabric Interface (plf) |
|---|---|---|---|---|---|---|---|---|---|---|
| 1+00 | 5.5 | 6.5 | 4.3 | 3.8 | 240 | 550 | 650 | 410 | 360 | 315 |
| 2+00 | 6.0 | 7.0 | 3.2 | 2.7 | 170 | 600 | 700 | 530 | 410 | 345 |
| 3+00 | 6.0 | 7.0 | 2.9 | 2.4 | 150 | 600 | 700 | 550 | 410 | 345 |
| 4+00 | 6.0 | 7.0 | 4.9 | 4.4 | 275 | 600 | 700 | 425 | 410 | 345 |
| 5+00 | 8.3 | 8.3 | 11.2 | 10.7 | 670 | *798 | *798 | 128 | 508 | 450 |
| 6+00 | 6.2 | 6.2 | 10.4 | 9.9 | 620 | 620 | 620 | ***0 | 330 | 355 |
| 7+00 | 6.9 | 6.9 | 2.4 | 1.9 | 120 | 690 | 690 | 570 | 400 | 400 |

* {8.3 ft - 0.8 ft (below tide)} 100 pcf + 0.8 ft (below tide)(60 pcf) = 798.

** 650 - (5.7)(50 psf) = 360 psf.

*** Complete bearing failure.

**** Computed as ($\gamma_s H \tan \phi_{sf}$) × (1 ft-width), $\phi_{sf} = 30^\circ$

TABLE X

EFFECTIVE SOIL STRESS IN FOUNDATION ALONG CENTER LINE OF EMBANKMENT FOUR MONTHS AFTER CONSTRUCTION

| Sta | Dike Height Above Fabric ($H_1$, ft) | Dike Height Above Foundation ($H_2$, ft) | Pore Pressure (el, ft) | Excess Pore Pressure Minus Tide 0.5 ft ($h_w$, ft) | Excess Pore Pressure $\gamma_w h_w$ (psf) | Total Stress On Fabric $\gamma_s H_1 = \sigma_{Tf}$ (psf) | Total Stress On Foundation $\gamma_s H_2 = \sigma_T$ (psf) | Effective Stress on Fabric, $\bar{\sigma} = \sigma_{Tf} - \gamma_w h_w$ (psf) | **Ultimate Bearing Capacity On Foundation $\sigma_T - 5.7c$ (psf) | Effective Stress Increase from Consolidation ($\Delta\bar{\sigma}$ psf) |
|---|---|---|---|---|---|---|---|---|---|---|
| 1+00 | 5.8 | 6.8 | 3.5 | 3.0 | 190 | 580 | 680 | 390 | 390 | 80 |
| 2+00 | 6.4 | 7.4 | 3.1 | 2.6 | 160 | 640 | 740 | 480 | 450 | 50 |
| 3+00 | 6.3 | 7.3 | 2.0 | 1.5 | 95 | 630 | 730 | 535 | 340 | 85 |
| 4+00 | 6.1 | 7.1 | 3.2 | 2.7 | 170 | 610 | 710 | 440 | 420 | 115 |
| 5+00 | 9.0 | 9.0 | 6.7 | 6.2 | 390 | *760 | *760 | 370 | 470 | 180 |
| 6+00 | 6.9 | 6.9 | 4.0 | 3.5 | 220 | 690 | 690 | 470 | 400 | 470 |
| 7+00 | 7.0 | 7.0 | 2.0 | 1.5 | 95 | 700 | 700 | 605 | 410 | 35 |

* {9.0 ft - (3.5 ft)(below tide)} 100 pcf + (3.5 ft)(60 pcf) = 760 psf.

** 680 psf - 5.8 (50 psf) = 390 psf.

and consolidation of the soft foundation materials. The original ground surface varied from about el 1.4 on the north abutment and el 0.9 on the south abutment to a low of about el 0.5 in the channel.

A total of about 1.0 to 1.5 ft of fill material was placed on the fabric prior to settlement plate installation and relatively small movements of the plates and fabric were evident until after construction reached sta 4+00 and the mud wave method of advancing the fabric was begun. At this point, the settlement plate and fabric elevations along the center line of the dike fell below el 0.0, but subsequently rose to approximately 1.3 ft or about el 2 as the mud wave moved up onto the south side of the channel.

As the embankment was constructed and completed, consolidation of the underlying soft foundation was recorded and is shown in the shaded area on the bottom of Figure 21. Maximum consolidation of 1.7 ft was observed at sta 6+00; sta 5+00 had the second highest value of consolidation at 1.6 ft. These consolidation values are in general agreement with the 3.0 ft predicted in Chapter IV.

Subsidence of crest height reflects consolidation of foundation materials. The crest, raised to a final grade of el 8, has exhibited a maximum settlement of about 1.3 ft. The 0.3- to 0.4-ft discrepancy between consolidation measurements on the fabric and crest subsidence readings is a function of the time lag between installation of the settlement plates, completion of the dike to grade, and possible further densification of the sand fill in the dike section.

## Horizontal Displacement

Horizontal displacements, plotted on an axis transverse to the dike alignment, are shown in a plan view of the embankment in Figure 79,

Appendix C. Most of the horizontal movement occurred immediately after the settlement plates were installed, which indicates that some transverse movement could have occurred during placement of the fabric and placement of the 1.5 to 2.0 ft of sand fill material on the fabric. Lateral spreading and vertical displacements were greatest on the east side of the embankment between sta 4+00 and 6+00. Support gained from the longitudinal piece of Polyfilter X beneath the Nicolon 66475 apparently provided a degree of biaxial resistance not afforded by the transverse strips of Nicolon 66475.

Maximum horizontal displacement, transverse to the dike alignment, caused fabric elongation to occur between the settlement plates located at sta E 0+06 and E 0+36* and at sta 5+00 and 6+00 with displacements of 1.2 and 1.1 ft, respectively. Percent fabric elongation determined by dividing the displacement (1.2 ft) by the transverse distance (30 ft) was 4.0 percent. There was not a large difference in the horizontal displacements between the settlement plates located at sta 00+36 and 00+58 (east and west side of center line). Dump trucks using the 22-ft space between these rows of settlement plates as a haul road may have caused the roadway to move laterally as one compacted unit. Also, the roadways contained two layers of fabric that provided additional strength and increased stiffness (modulus) that would contribute to this type of behavior. Therefore, the use of two parallel haul roads near the embankment toe caused stretching of the fabric as was hypothesized in Chapter IV.

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* The station prefix E indicates distance east of the longitudinal embankment center line.

By plotting the maximum percent elongation obtained under field conditions (4.0 percent) on Figure 15, Chapter IV, which shows the stress-strain data obtained for the five geotechnical fabrics that met the desired tensile strength, the fabric tensile stress actually developed in the Nicolon 66475 fabric was determined to be about T = 85 lb/in.-width or 1000 lb/ft-width.

## Comparisons of Fabric Stress Measured in the Field and Fabric Stress Predicted by Analytical Procedures

An analysis of the data collected during this study was conducted by comparing the fabric stress measured in the field at the end of construction with the fabric stress predicted by analytical procedures discussed earlier in this report and in Appendix D. The four potential unsatisfactory modes of behavior postulated for fabric-reinforced embankments were (a) horizontal/lateral spreading or sliding of the embankment, (b) local bearing failure and rotational subsidence of the soft foundation, (c) embankment failure by excessive deformation before stable bearing conditions are achieved, and (d) insufficient fabric anchorage during embankment deformation.

### Horizontal Sliding

Resistance to horizontal sliding criteria assumed that, although the soil-fabric frictional resistance of the embankment may be sufficiently greater than the lateral earth pressure necessary to cause sliding, the tensile strength of the fabric may not be great enough and failure may result in fabric tearing and outward sliding of the embankment along the soft foundation.

As discussed in Chapter IV, the horizontal force that might cause lateral sliding was approximated by Mohr-Coulomb active pressure. The lateral load calculated for the end of construction is:

$$P_a = 0.5\ \gamma_s H^2 \tan^2 (45^\circ - \frac{\phi}{2})$$

where $\gamma_s$ = density of embankment sand, 100 pcf

$H$ = embankment height at sta 5+00 at end of construction, 8.3 ft

$\phi$ = frictional resistance of embankment sand, $30^\circ$

or

$$P_a = 0.5\ (100\ \text{pcf})(8.3\ \text{ft})^2 \tan^2 (45 - \frac{\phi}{2})$$

$$P_a = 1150\ \text{lb/ft-width}$$

The horizontal sliding resistance necessary to resist the active pressure would be the ultimate stress of the fabric. Observations made during construction and inspection of the vertical and horizontal settlement plate data in Appendix C indicated that horizontal sliding had occurred with a fabric elongation of about 4.0 percent and a fabric tensile stress of about 1000 lb/ft-width for the Nicolon 66475 fabric. If a minimum safety factor of 2.0 is chosen against sliding, the fabric would provide an ultimate tensile strength of 2.0 × 1150 lb/ft-width or 2300 lb/ft-width, which would exceed the measured tensile stress of 1000 lb/ft-width. Results of these calculations are shown tabulated in Table XI. This very close agreement of measured and calculated tensile stress indicates that this potential unsatisfactory mode controlled the sliding behavior of the test section.

TABLE XI

CALCULATED FABRIC STRENGTHS NECESSARY TO PREVENT EMBANKMENT FAILURE

| Design Method | Factor of Safety FS | Fabric Strength | | Fabric Elongation Nicolon 66475 | †Factor of Safety Against Failure Measured Data (FS) |
|---|---|---|---|---|---|
| | | lb/ft-width | lb/in.-width | | |
| * Sliding | 1.0 | 1150 | 95 | 4.9 | 1.1 |
| Wedge | 2.0 | 2300 | 190 | 7.0 | 2.2 |
| ** Modified Bishop | 1.0 | 2040 | 170 | 7.5 | 2.0 |
| Method | 1.3 | 4090 | 340 | 10.0 | 4.0 |
| (circular arc) | | | | | |
| * Bearing | 1.0 | 540 | 45 | 2.1 | 0.5 |
| Failure | 3.0 | 1620 | 140 | 6.5 | 1.7 |
| q = 5.7c | | | | | |

* 8.3 ft
** 8.0 ft design curves, Appendix D
† Factor of safety against failure: fabric design strength divided by fabric measured strength (95 lb/in.-width ÷ 85 lb/in.-width = 1.1).

Local Bearing Failure and

Rotational Subsidence

Local bearing failure resulting from localized foundation failure with a sliding/slumping of the embankment such as a circular arc rotation through the toe of the dike was investigated by use of the modified Bishop Method. This method of analysis was used to estimate the fabric ultimate tensile strength and to provide a factor of safety against rotational slope failure in the soft clay foundation materials for a dike height of H = 8 ft. Results of this investigation indicated that the fabric ultimate tensile strength required to prevent failure was 2040 lb/ft-width at a factor of safety of 1.0. This fabric strength requirement is twice as large as the fabric strength required to resist the horizontal sliding mode, but is only about half of the actual fabric stresses measured. It was recommended earlier in Chapter IV that a minimum factor of safety, between 1.1 and 1.2, would prevent rotational subsidence, but because this behavior is one of the most difficult to measure, a factor of safety of 1.3 would be more conservative and the chances of success more probable. These data are shown tabulated in Table XI.

There was no evidence of sliding or slumping that might have resulted in a localized bearing failure or stress concentration in the fabric in the embankment test section. A circular arc rotational type failure that resulted in deformation embankment and deformation in the fabric at the point of sliding was observed in a test section constructed in Holland and reported by Risseeuw.[4] This type of failure was

documented and data supporting this type of failure were provided from the author and manufacturers, who support these findings.

Until field data from controlled tests or prototype structures of this type of behavioral mode become available, it would be expedient to use the fabric strengths determined by the modified Bishop method of analysis. Identifying and measuring the stress in the fabric where these rotational failures may occur, especially at localized points of possible high fabric stress concentration, is very important to these analyses, and every effort should be made to document this type of potentially unsatisfactory behavior in future projects. However, based on observed behavior for the test embankment, classic slope stability analysis overpredicts the needed fabric strength by a factor of about 170/85 or 2. Thus, this assumed mode of failure was not critical for the test section.

## Fabric Tensile Stress Developed by Embankment Deformation

It was postulated in Chapter IV that, once the foundation bearing capacity was exceeded by the embankment bearing pressure, then bearing failure and resulting deformation of the foundation would occur. To avoid this type of failure, insofar as possible, the fabric was placed, covered, and anchored by the embankment material before excessive deformations could occur. Bearing capacity of the foundation was exceeded when the embankment height exceeded 3 ft or about 290 psf, and it was assumed that the fabric would carry the remaining weight of dike (830 psf - 290 psf or 540 psf) and the embankment would tend to slide or spread laterally, causing tension stresses in the fabric.

Effective soil stresses determined from piezometers along the center line of the embankment at the end of construction were shown in Table IX. The minimum effective stresses determined at sta 5+00 and 6+00 were 110 and 0 psf, respectively, which were small and confirmed the rationale of using unconsolidated undrained shear strength for ultimate bearing capacity calculations.

If foundation bearing failure occurred, the frictional force caused by incipient embankment sliding must be carried with the fabric. This stress was calculated by multiplying the weight of the embankment by the tangent of the angle of internal friction and values are shown in Table IX for end-of-construction conditions. The maximum horizontal stress was 450 lb/ft-width at sta 4+00 or about half the measured stress.

In any case, use of the bearing capacity method for determining the required fabric strength to resist the static loads of the embankment was unsatisfactory.

## Fabric Anchorage Failure

Although this failure mode had been postulated, no method of predictive analysis was applied, and excavation and observation of the fabric foldback in the dike toe indicated that the foldback section of fabric was under no noticeable stress. This failure mode may have significance for embankments with steep side slopes, but it apparently is not critical for shallow-sloped embankments.

Earlier in the report it was estimated that approximately 3 ft of consolidation would occur near the center of the dike section. If this is assumed to be true and no lateral displacement is allowed, then the percent fabric elongation or, consequently, fabric stress can be determined

geometrically from the curve in the discussion in Chapter III. The percent elongation is calculated from the following expression:

$$\varepsilon = \sqrt{\left(\frac{H}{L}\right)^2 + 1} - 1$$

where H = embankment height, 8 ft

L = one half of the foundation base width

3 ft of consolidation = 3/8 H

then, the percent elongation is as follows:

$$= \sqrt{\frac{3/8(8)}{76} + 1} - 1$$

$$\varepsilon = 0.042$$

Therefore, it can be concluded that an average elongation of this magnitude will not produce appreciably large stresses in the fabric, thus minimal end anchorage was necessary.

*Summary of Analysis*

Analysis based on field observations and design strengths determined by various design procedures confirmed the need for fabric for reinforcement to prevent embankment failure. Maximum fabric elongation of 4 percent (strain) at 85 lb/in.-width or 1000 lb/ft-width was recorded in the Nicolon 66475 fabric at sta 5+00 in the test section. A fabric tensile stress of 85 lb/in.-width would have provided the percent fabric elongations for the fabrics shown in Table XII.

These data indicate that fabric elongation at the stress experienced by the Nicolon 66475 was within the maximum elongation allowed for fabric selection during the fabric tests. Earlier in Chapter IV, it was stated that fabric elongation of less than 3 to 5 percent was desired, but fabric elongations of 10 percent would be accepted in the test

TABLE XII

FABRIC ELONGATION

| Fabric | Percent Elongation at 85 lb/in.-width or 1000 lb/ft-width |
|---|---|
| Nicolon 66475 | 4.0 |
| Nicolon 66186 | 7.5 |
| Advance Type I | 8.0 |
| Polyfilter X | 9.0 |
| Bay Mills | 3.3 (not used in test section) |

section. Comparison of other fabrics was considered to be unnecessary because moduli determined for these fabrics was much less than those used in the test section and excessive fabric elongation would have been prohibitive at this fabric stress.

Comparison of the design procedure used in this analysis indicated that the sliding wedge analysis is more appropriate in that the fabric stress determined by this method was almost identical to the fabric stress measured in the field. The modified Bishop method was more conservative, predicting a fabric stress of approximately double that measured in the field, whereas the bearing failure method predicted a value of about one half the field measurement.

Fabric elongation caused by vertical foundation displacement or consolidation was minimal. There was no evidence of any benefit from anchoring the fabric by folding the outer edge into the toe except for improved truck mobility by the double fabric layers in the parallel haul roads.

## ENDNOTES

[1]Haliburton Associates, "Design of Test Section for Pinto Pass Dike, Mobile, Alabama" (Stillwater, Oklahoma, 1978). Prepared under Contract DACW01-78-C-0092 for the U. S. Army Engineer District, Mobile. pp. 44-49.

[2]Ibid., p. 45.

[3]Ibid., p. 47.

[4]P. Risseeuw, "Stabilenka Woven Reinforced Fabric in Raising Mounds of Soft Soil" (Unpublished report, Akzo Research Laboratories, Arnhem, The Netherlands, 1977).

# CHAPTER VI

# CONCLUSIONS AND RECOMMENDED DESIGN AND CONSTRUCTION CONSIDERATIONS

## Conclusions

It is concluded that concepts controlling the design, construction, and evaluation of fabric-reinforced earth embankments constructed on very soft foundation material have been well defined in this research investigation. Data obtained from this study were used to develop design and construction techniques for this type of construction. Collection and evaluation of these data have verified the technical feasibility of the concepts and the applicability of these concepts for the continued construction of fabric-reinforced embankments at Pinto Pass or in similar future dike construction on soft foundations. Therefore, it is concluded that methods for proper design and construction have been developed, and the factors concerning design, construction, and analysis for estimating the tensile stresses developed in the fabric as a result of embankment deformation have been clearly understood.

## Recommended Design and Construction Considerations

An assessment and evaluation of the construction project at Pinto Pass revealed several areas where activities could be optimized for future construction.

A summary of design construction considerations and recommendations were evaluated as to their importance or lack of importance in fabric-reinforced embankments constructed on soft foundation and are listed in the following sections.

## Important Design Considerations and Recommendations

1. Prior design considerations should evaluate three potential failure modes: lateral spreading or sliding failure, rotational failure, and bearing failure. Anchorage considerations do not appear important for flat slope embankments.

2. Criteria for fabric selection should include high strength-low elongation (fabric with high modulus or less than 4 percent elongation at working load), low creep under load, uniaxial fabric strength, wet strength properties, corrosion resistance to various elements found in the environment, and ultraviolet resistance prior to installation.

3. Three geometrical parameters that should be considered during embankment design are embankment height H; embankment slope $\beta$, and depth of soft foundation layer h.

4. Three soil parameters important in design for end of construction conditions are density $\gamma$, cohesion c, and internal angle of friction $\phi$, for both fill and foundation materials. Unconsolidated-undrained Q triaxial compression or shear test are recommended for the cohesive soils and for cohesionless direct shear test of fill material in a relatively loose condition.

5. Geotechnical exploration is required to describe foundation conditions and to obtain samples for laboratory testing.

6. The cost of placing 1 sq yd of fabric is less than or equal to placing 1 cu yd of fill material, thus fabric-reinforced embankments may be considerably more cost-effective than displacement sections.

7. Timing for sequential dike raising increments should be based on the decrease in pore water pressure in the foundation. Additional incremental dike construction could have begun 6 to 7 months after construction of Pinto Pass.

## Important Construction Considerations and Recommendations

1. Fabric edge seams should always be oriented transverse to the longitudinal axis of the embankment (fabric warp direction transverse to alignment).

2. Sequential construction operations that made use of two parallel haul roads at each toe of the dike should be used to allow control of horizontal displacement, control of fabric stress caused by initial bearing displacement, and controlled placement of fill material to cause prestressing of fabric between parallel haul roads.

3. Use of low-ground-pressure dozers and lightweight haul equipment such as small tandem-wheel dump trucks in good mechanical condition (with experienced operators) is of vital importance to successful construction.[1,2]

4. Items that should also be considered include: workability of the fabric, as to whether the fabric is hydrophobic or hydrophillic; the relative ease to place and sew; and performance relative to expediting the installation. Also, when placed over soft foundation material, the stiffness of the fabric should be adequate to support workers.

5. Fabric strips should be continuous from toe to toe of the embankment, without transverse seams.

6. Good sewing techniques should include a proper sewing machine, proper thread size (bonded No. 12 nylon, 100-lb tensile breaking strength), and proper number of sewing passes per seam (not less than two).

7. Double-layer fabric (from foldback) haul roads should be used. These received a large number of repetitions without excessive rutting or failure and were considered beneficial in hauling operations.

8. Fabric should provide a good separation between the fill material and the foundation material.

9. Keeping good records of fabric usage and installation location is important for future evaluation of fabric performance.

10. Type of construction contract required to build the prototype structure should be an equipment rental contract.

## Less Important Design Considerations

1. Exact knowledge of the foundation soil conditions may not always be necessary if proper worst-case design is conducted.

2. Vertical foundation short-term displacement and long-term consolidation does not cause significant fabric elongation.

3. Foundation thickness may be used to determine amount of consolidation that may occur, but has no major effect on fabric-reinforced embankment stability.

4. Excess pore pressures sufficient to cause zero effective soil pressure $\bar{\sigma}$ from embankment loading do not affect fabric stress requirements for $\phi = 0^{o}$ material. Design using foundation unconsolidated-undrained Q strength is appropriate.

5. Fabric permeability is not too important because of low permeability of foundation soil.

6. Biaxial fabric loads are not important for design purposes but may play a small role during construction if placement causes a mud wave and stretching of the fabric strips transverse to their long axis.

7. Fabric properties that do not play an important role in design or construction are temperature susceptibility, abrasion, roughness (texture), burst strength, filtration (EOS), and thickness.

Less Important Construction Considerations

1. Construction of a working table prior to fabric placement is not important as long as the surface is reasonably flat and not too rough.

2. Fabric wrinkling, minimized by selective placement of fill material to straighten and smooth out wrinkles, is not considered to be a major problem.

3. Fabric folded back at the embankment toes did not appear to be stressed at the fold back edge; therefore, benefit from anchorage was considered to be negligible for this embankment with shallow slopes, but could be a problem for steep slopes.

4. Weather conditions did not present any major problems during construction but adverse weather such as heavy rains, severe cold, or high tides could cause delays in construction for short periods of time.

## General Recommendations

Pending availability of results from other similar construction, future fabric-reinforced embankment construction at Pinto Pass and elsewhere

should be designed to prevent lateral spreading because loading developed in the test section approximated predicted stresses. Recommended factor of safety for lateral spreading and bearing failure is FS = 2.0; for slope stability analysis (Modified Bishop method), a factor of safety of 1.3 should be used to determine fabric strength. It is recommended that the minimum stress-strain modulus of the fabric used be such that no more than 4 percent fabric elongation is developed under working stresses.

It is also recommended that the fabric foldback construction procedure at each toe of the embankment be eliminated and adequate truck support be provided by placing a strip of fabric wide enough to carry a dump truck (15 ft) along the toe parallel with the dike alignment, with about 1 ft of sand between the transverse dike reinforcement and the parallel upper strip. This procedure will provide a double fabric-reinforced haul road without causing a construction bottleneck. It is also recommended that further research be conducted to determine proper sewing techniques and thread requirements.

ENDNOTES

[1]Willoughby, W. E., "Low Ground-Pressure Construction Equipment for Use in Dredged Material Containment Area Operation and Maintenance: Performance Predictions," Technical Report D-77-7 (U. S. Army Engineer Waterways Experiment Station, CE, Vicksburg, Mississippi, 1971). NTIS No. AD A044 209.

[2]Willoughby, W. E., "Assessment of Low Ground-Pressure Equipment for Use in Containment Area Operation and Maintenance," Technical Report DS-78-9 (U. S. Army Engineer Waterways Experiment Station, CE, Vicksburg, Mississippi, 1978). NTIS No. AD A058 501.

## APPENDIX A

## CONVENTIONAL SOIL TESTING

Drilling, collection, and classification of soil samples for the field and laboratory tests were carried out by personnel of the Foundation and Materials Branch, MDO, and personnel from the WES. Collection of undisturbed samples and field vane shear tests, conducted by WES personnel, were made possible through the use of the Riverine Utility Craft (RUC). Field borings conducted by the MDO were made from the Pinto Pass fabric-reinforced dike section during and after construction and were advanced to a maximum depth of about 40 ft with a lightweight drill rig mounted on a swamp buggy. Soil samples collected from the RUC were taken with a hand-held 1.85-in.-diam. thin-wall Hvorslev tube sampler to a maximum depth of about 17 ft. Collection of undisturbed soil samples, split-spoon samples, and recording Standard Penetration Test N-values and soil classification were conducted during operations by the MDO. Conventional laboratory tests conducted on split-spoon soil samples included visual classification and water-content determinations.

Specific gravity tests, triaxial unconsolidated-undrained Q tests and consolidated-undrained R tests, and one consolidation test were performed on undisturbed samples taken from Shelby tubes and Hvorslev sample tubes. Atterberg limits were determined for the clay samples, and a sieve analysis was performed on the sand used as fill material for dike construction.

Results of the laboratory tests are shown in Table XIII. The soil in Pinto Pass was a brown to black plastic clay (CH) to a depth of 15 ft on the west end of the pass and to a depth of 40 ft on the east end of the pass with clayey to silty sand lenses and stringers intermingled throughout until a clean, dense white sand was penetrated at lower depths.

A plan view of the borehole layout for the west end of Pinto Pass along the longitudinal axis of the 800-ft test section is shown in Figure 22, and a soil boring legend is shown in Figure 23. Standard Penetration Test N-values, shown along with boring logs and water content determinations in Figures 24 through 27, were zero or the weight of the hammer in the plastic clay zones, somewhat higher than zero in the silty and clayey sand layers, and very high in the dense white sand. A profile view of soil beneath the embankment is shown in Figure 28. A gradation curve for the sand fill material borrowed from dredged material containment areas used to construct the embankment is shown in Figure 29.

Vane shear tests conducted by personnel from WES indicated that unconsolidated-undrained shear strengths for the plastic clay in the west end of the pass was about 50 psf to a depth of 10 ft and about 100 psf for the next 5 to 7 ft, which was the limit of the testing device.

The following ranges of soil properties were determined for the plastic clay soil properties to a depth of about 15 to 17 ft MSL:

| | |
|---|---|
| Specific gravity | 2.72-2.74 |
| Water content, percent | 46-122 |
| Dry unit weight, pcf | 39-59 |
| Void ratio | 1.28-3.41 |

| | |
|---|---|
| Liquid limit | 65-101 |
| Plastic limit | 23-24 |
| Plasticity index | 42-67 |
| Degree of saturation, percent | 98-100 |

The plasticity index versus liquid limit of the samples from the undisturbed borings used in the triaxial compression tests are shown in Figure 30.

Two undisturbed soil specimens of plastic clay from depths of 5 to 12 ft were obtained to determine the unconsolidated-undrained, Q-test strength of the foundation materials. Specimens were nominally 1.4 in. diam and 3.0 in. high. Data from these tests are shown in Figures 31 and 32 and tabulated in Table XIII. The approximate shear strength and/or cohesion c ranged from 0.03 to 0.11 tsf from 5- to 12-ft depths, respectively, and the angle of internal friction was zero.

A total of three consolidated-undrained R-tests were performed on undisturbed soil specimens from depths varying from 5 to 12 ft. The samples were nominally 1.4 in. diam and 3.0 in. high. Data from these tests are shown in Figures 33 through 38 and tabulated in Table XIII. Cohesion c before consolidation varied from 0.07 to 0.11 tsf from depths of 5 to 12 ft, respectively, and the angle of internal friction varied from 10 to 16° for the same depths. The cohesion c after consolidation varied from 0.12 to 0.15 tsf and the angle of internal friction varied from 22 to 31° for 5- and 12-ft depths, respectively. The shear strength determined from the unconsolidated-undrained tests varied from 0.22 to 0.62 tsf for depths of 5 to 12 ft, respectively. The final back pressure for the samples tested from the 5-ft depth was 2.88 tsf and for samples from 10.5- to 12.0-ft depths, the final back pressure was 3.6 tsf.

Consolidation tests were conducted on one sample obtained from a depth of 9.0 to 12.0 ft. The e-log p curve was slightly concave upward and to the right, indicating that the clay might be slightly sensitive or deformation causing this shaped curve might be due to rearrangement of grains because there was little rebound after release of load. The compression index $C_c$ was determined to be about 1.0 from the curve shown in Figure 39.

Additional data obtained during and after the feasibility study include a soil profile view of the foundation materials for the proposed dike alignment near the east end of Pinto Pass. These data are shown in Figure 40.

TABLE XIII

TRIAXIAL TESTS

| Borehole Location | Depth el ft MSL | Material Description | Approximate Dry Unit Wt, pcf | Approximate Water Content % | Angle of Friction Ø, deg | Cohesion C, tsf | Approximate Average Overburden Pressure tsf | Chamber Pressure $\sigma_3$ tsf | Final Back Pressure tsf | Compressive Strength $(\sigma_1 - \sigma_3)$ tsf | Approximate Shear Strength tsf |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | **Q Tests** | | | | | | |
| East Dike ℄ | -5.0 to -5.5 | Plastic clay (CH) brownish gray and black | 38 | 130 | 0 | 0.03 | 0.22 | 0.25 | 0.25 | 0.05 | 0.03 |
| | | | | | | | | 0.50 | | 0.08 | 0.04 |
| | | | | | | | | 1.00 | | 0.07 | 0.04 |
| West Dike ℄ | -9.0 to -12.0 | Plastic clay (CH) blackish gray | 56 | 76 | 0 | 0.10 | 0.52 | 0.30 | | 0.22 | 0.11 |
| | | | | | | | | 0.50 | | 0.20 | 0.10 |
| | | | | | | | | 1.00 | | 0.20 | 0.10 |
| | | | | | **R Tests** | | | | | | |
| East Dike ℄ | -5.0 to -5.5 | Plastic clay (CH) brownish gray and black | 45 | 103 | $\underline{R}$ 13 | $\underline{R}$ 0.07 | 0.23 | 0.50 | 2.88 | 0.45 | 0.22 |
| | | | | | $\bar{R}$ 25 | $\bar{R}$ 0.12 | | 1.0 | 2.88 | 0.55 | 0.34 |
| | | | | | | | | 1.50 | 2.88 | 1.02 | 0.50 |
| West Dike ℄ | -5.0 to -5.5 | Plastic clay (CH) brownish gray and black | 44 | 104 | $\underline{R}$ 10 | $\underline{R}$ 0.11 | 0.22 | 0.5 | 2.88 | 0.45 | 0.22 |
| | | | | | $\bar{R}$ 22 | $\bar{R}$ 0.15 | | 0.75 | 2.88 | 0.69 | 0.30 |
| | | | | | | | | 1.00 | 2.88 | 0.72 | 0.36 |
| | | | | | | | | 1.50 | 2.88 | 0.91 | 0.45 |
| West Dike ℄ | -10.5 to -12.0 | Plastic clay (CH) blackish gray; organic matter | 62 | 67 | $\underline{R}$ 16 | $\underline{R}$ 0.07 | 0.57 | 0.5 | 3.6 | 0.77 | 0.28 |
| | | | | | $\bar{R}$ 31 | $\bar{R}$ 0.15 | | 1.0 | 3 6 | 0.64 | 0.42 |
| | | | | | | | | 1.5 | 3.6 | 1.27 | 0.62 |

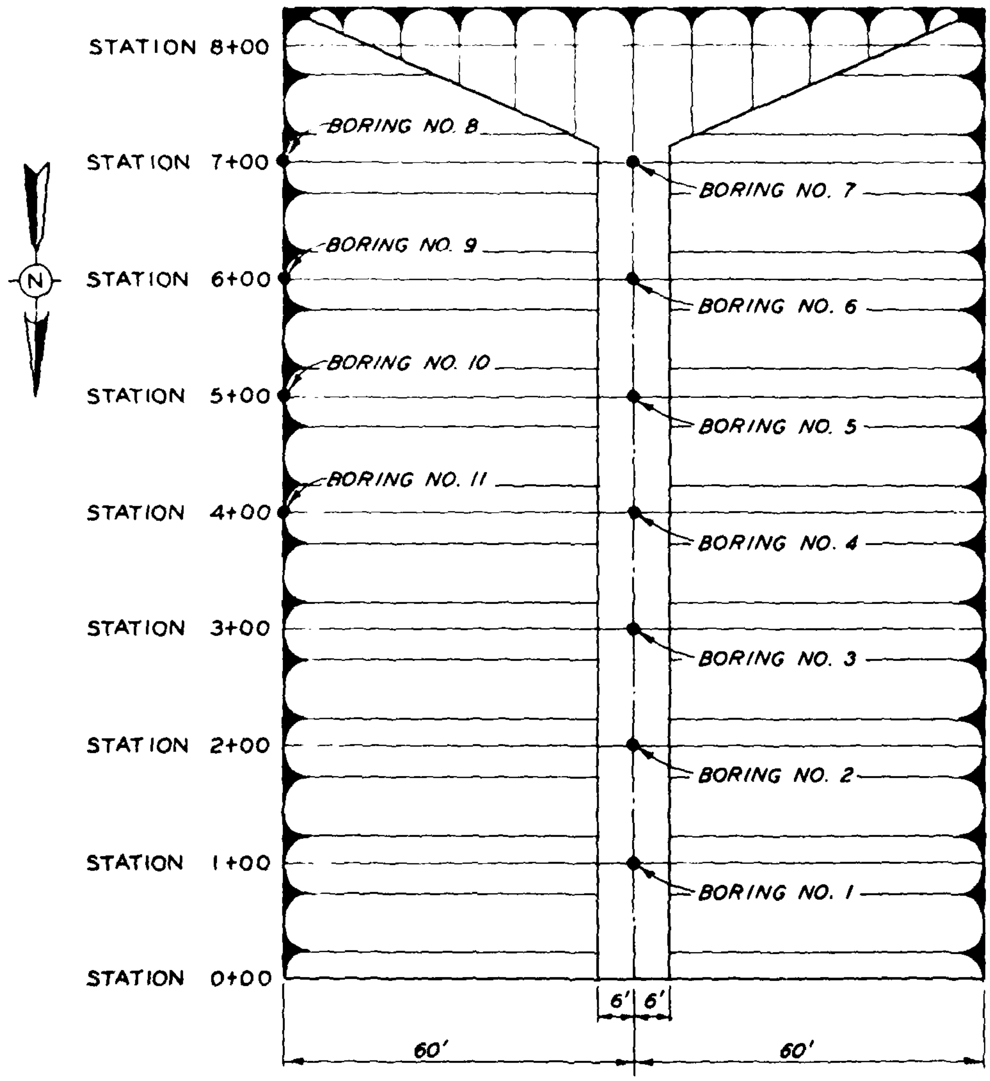



Figure 22. Borehole Layout Along Longitudinal Axis of the Test Section

## UNIFIED SOIL CLASSIFICATION

| MAJOR DIVISION | | TYPE | LETTER SYMBOL | SYMBOL | TYPICAL NAMES |
|---|---|---|---|---|---|
| COARSE GRAINED SOILS (More than half of material is larger than No. 200 sieve size) | GRAVELS (More than half of coarse fraction is larger than No. 4 sieve size) | CLEAN GRAVEL (Little or No Fines) | GW | | GRAVEL, Well Graded, gravel-sand mixtures, little or no fines |
| | | | GP | | GRAVEL, Poorly Graded, gravel-sand mixtures, little or no fines |
| | | GRAVEL WITH FINES (Appreciable Amount of Fines) | GM | | SILTY GRAVEL, gravel-sand-silt mixtures |
| | | | GC | | CLAYEY GRAVEL, gravel-sand-clay mixtures |
| | SANDS (More than half of coarse fraction is smaller than No. 4 sieve size) | CLEAN SAND (Little or No Fines) | SW | | SAND, Well-Graded, gravelly sands |
| | | | SP | | SAND, Poorly-Graded, gravelly sands |
| | | SANDS WITH FINES (Appreciable Amount of Fines) | SM | | SILTY SAND, sand-silt mixtures |
| | | | SC | | CLAYEY SAND, sand-clay mixtures |
| FINE GRAINED SOILS (More than half the material is smaller than No. 200 sieve size) | | SILTS AND CLAYS (Liquid Limit < 50) | ML | | SILT & very fine sand, silty or clayey fine sand or clayey silt with slight plasticity |
| | | | CL | | LEAN CLAY, Sandy Clay, Silty Clay, of low to medium plasticity |
| | | | OL | | ORGANIC SILTS and organic silty clays of low plasticity |
| | | SILTS AND CLAYS (Liquid Limit > 50) | MH | | SILT, fine sandy or silty soil with high plasticity |
| | | | CH | | FAT CLAY, inorganic clay of high plasticity |
| | | | OH | | ORGANIC CLAYS of medium to high plasticity, organic silts |
| HIGHLY ORGANIC SOILS | | | Pt | | PEAT, and other highly organic soil |
| WOOD | | | Wd | | WOOD |
| SHELLS | | | SI | | SHELLS |
| NO SAMPLE | | | | | |
| | | | Sh | | SHALE |

NOTE Soils possessing characteristics of two groups are designated by combinations of group symbols

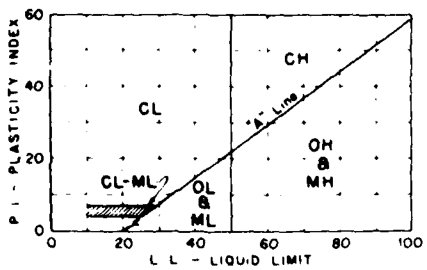
## DESCRIPTIVE SYMBOLS

**COLOR**

| COLOR | SYMBOL |
|---|---|
| TAN | T |
| YELLOW | Y |
| RED | R |
| BLACK | BK |
| GRAY | Gr |
| LIGHT GRAY | lGr |
| DARK GRAY | dGr |
| BROWN | Br |
| LIGHT BROWN | lBr |
| DARK BROWN | dBr |
| BROWNISH-GRAY | br Gr |
| GRAYISH-BROWN | gy Br |
| GREENISH-GRAY | gn Gr |
| GRAYISH-GREEN | gy Gn |
| GREEN | Gn |
| BLUE | Bl |
| BLUE-GREEN | Bl Gn |
| WHITE | Wh |
| MOTTLED | Mot |

**CONSISTENCY FOR COHESIVE SOILS**

| CONSISTENCY | COHESION IN LBS/SQ FT FROM UNCONFINED COMPRESSION TEST | SYMBOL |
|---|---|---|
| VERY SOFT | < 250 | vSo |
| SOFT | 250 - 500 | So |
| MEDIUM | 500 - 1000 | M |
| STIFF | 1000 - 2000 | St |
| VERY STIFF | 2000 - 4000 | vSt |
| HARD | > 4000 | H |




**PLASTICITY CHART**

For classification of fine-grained soils

**MODIFICATIONS**

| MODIFICATION | SYMBOL |
|---|---|
| Traces | Tr |
| Fine | F |
| Medium | M |
| Coarse | C |
| Concretions | cc |
| Rootlets | rt |
| Lignite fragments | lg |
| Shale fragments | sh |
| Sandstone fragments | sds |
| Shell fragments | slf |
| Organic matter | O |
| Clay strata or lenses | CS |
| Silt strata or lenses | SIS |
| Sand strata or lenses | SS |
| Sandy | S |
| Gravelly | G |
| Boulders | B |
| Slickensides | SL |
| Wood | wd |
| Oxidized | Ox |
| Crumbly | Cr |
| Loose | Lo |
| Vegetation | Veg |

Figure 23. Legend for Boring Logs (Figures 24-27)

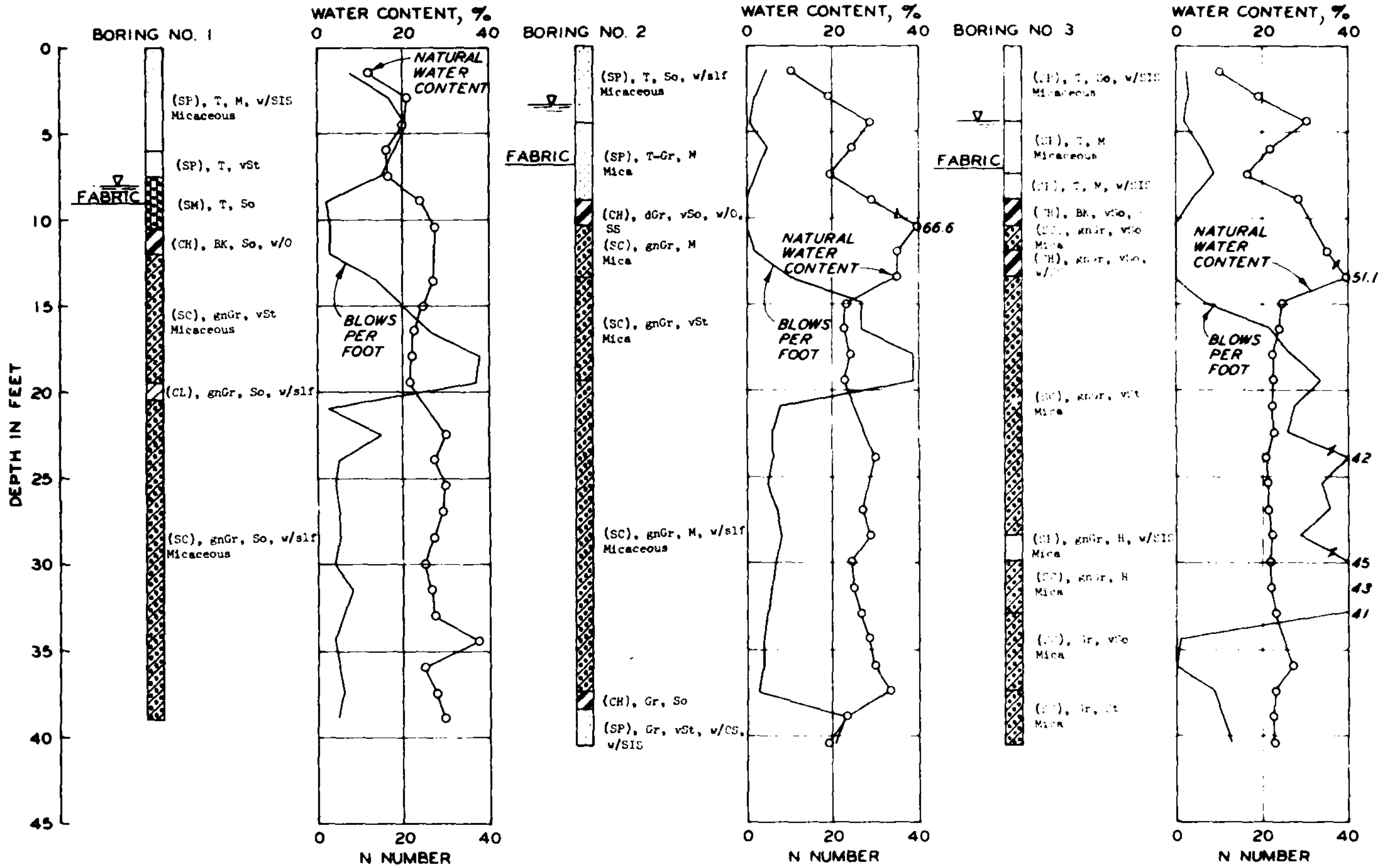



Figure 24. Boring Log and Water Content and N-Number versus Depth for Borings 1, 2, and 3

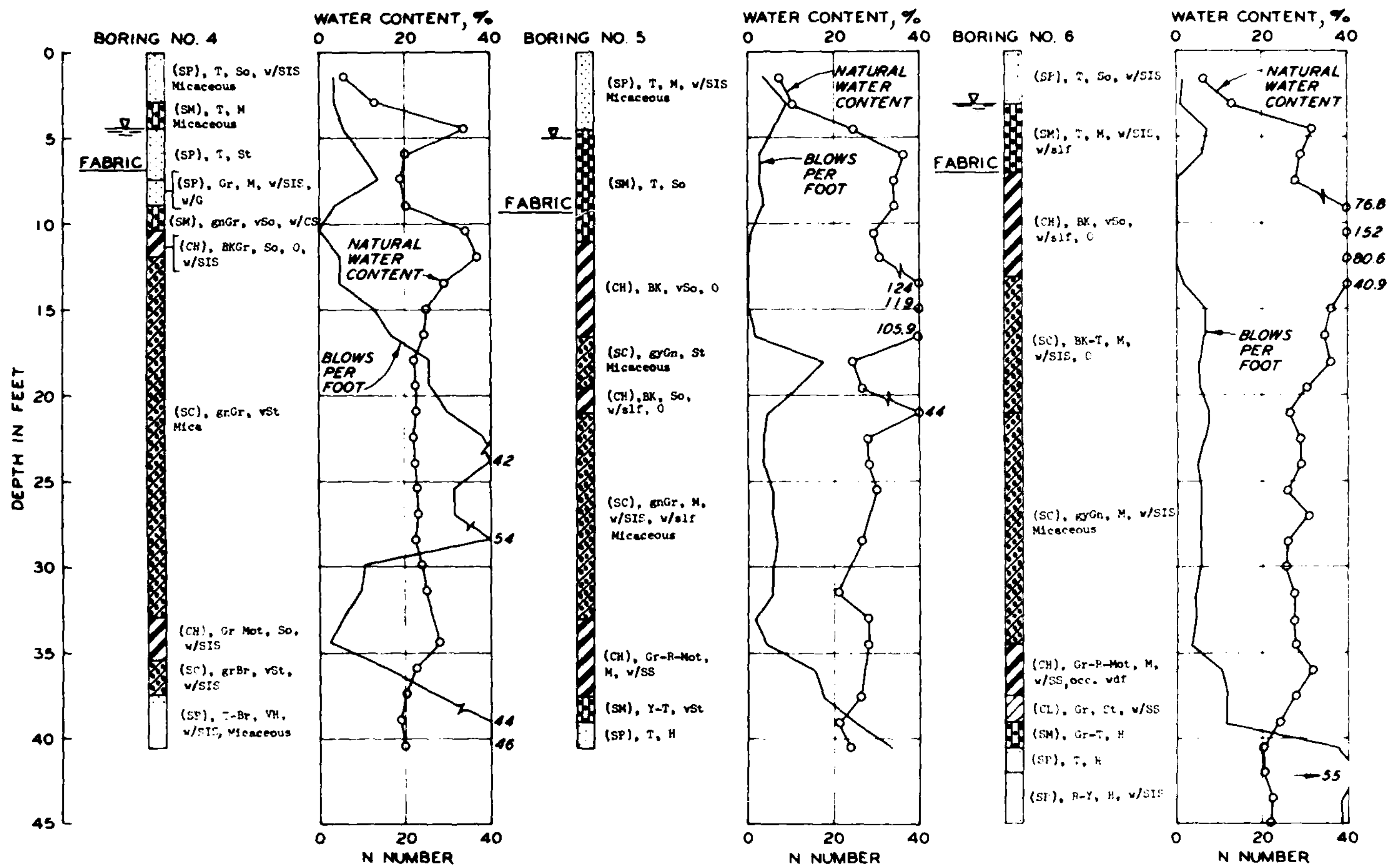



Figure 25. Boring Log and Water Content and N-Number versus Depth for Borings 4, 5, and 6

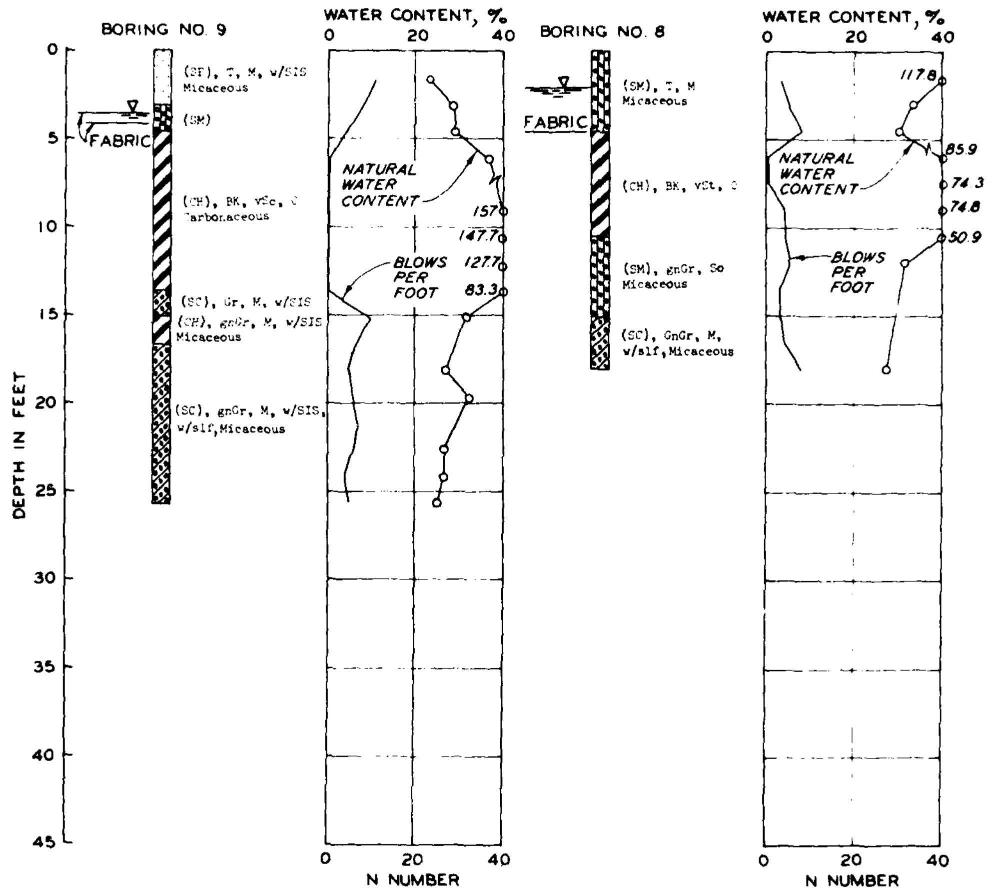



Figure 26. Boring Log and Water Content and N-Number versus Depth for Borings 8 and 9

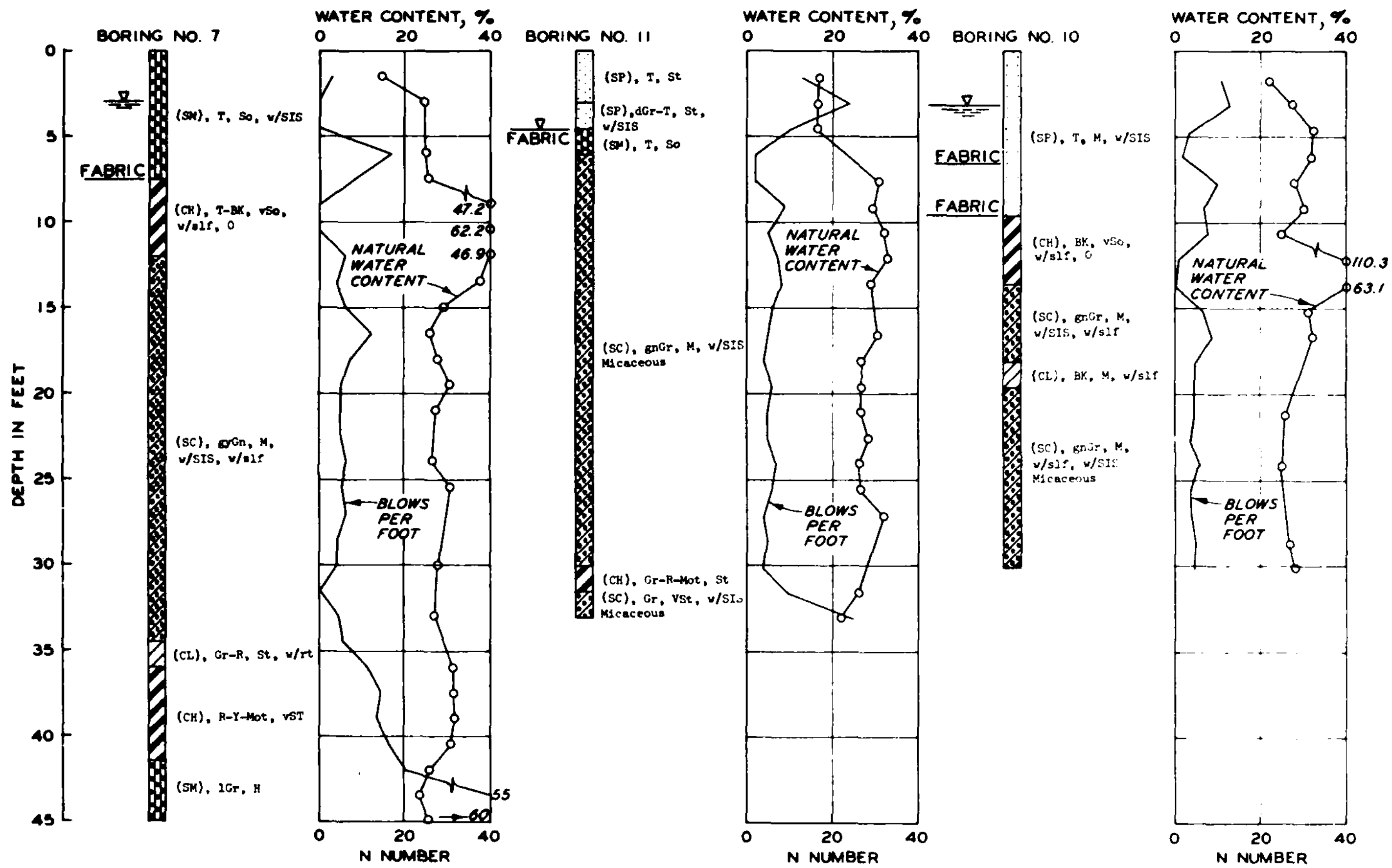
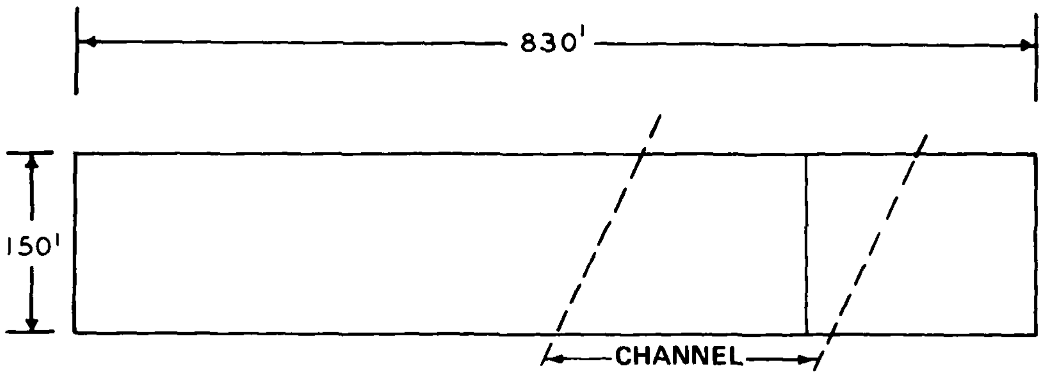
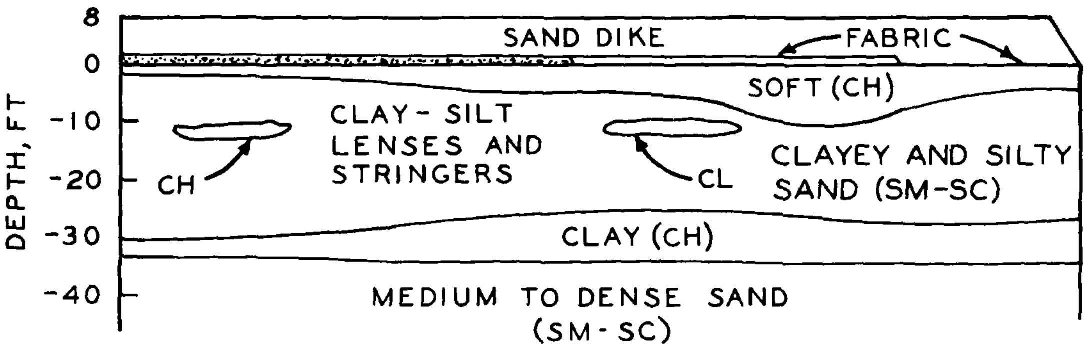



Figure 27. Boring Log and Water Content and N-Number versus Depth for Borings 7, 10, and 11




A. PLAN VIEW OF EMBANKMENT TEST SECTION




B. TYPICAL SOIL PROFILE, PINTO PASS, ALABAMA

Figure 28. Plan and Soil Profile, Embankment Test Secion

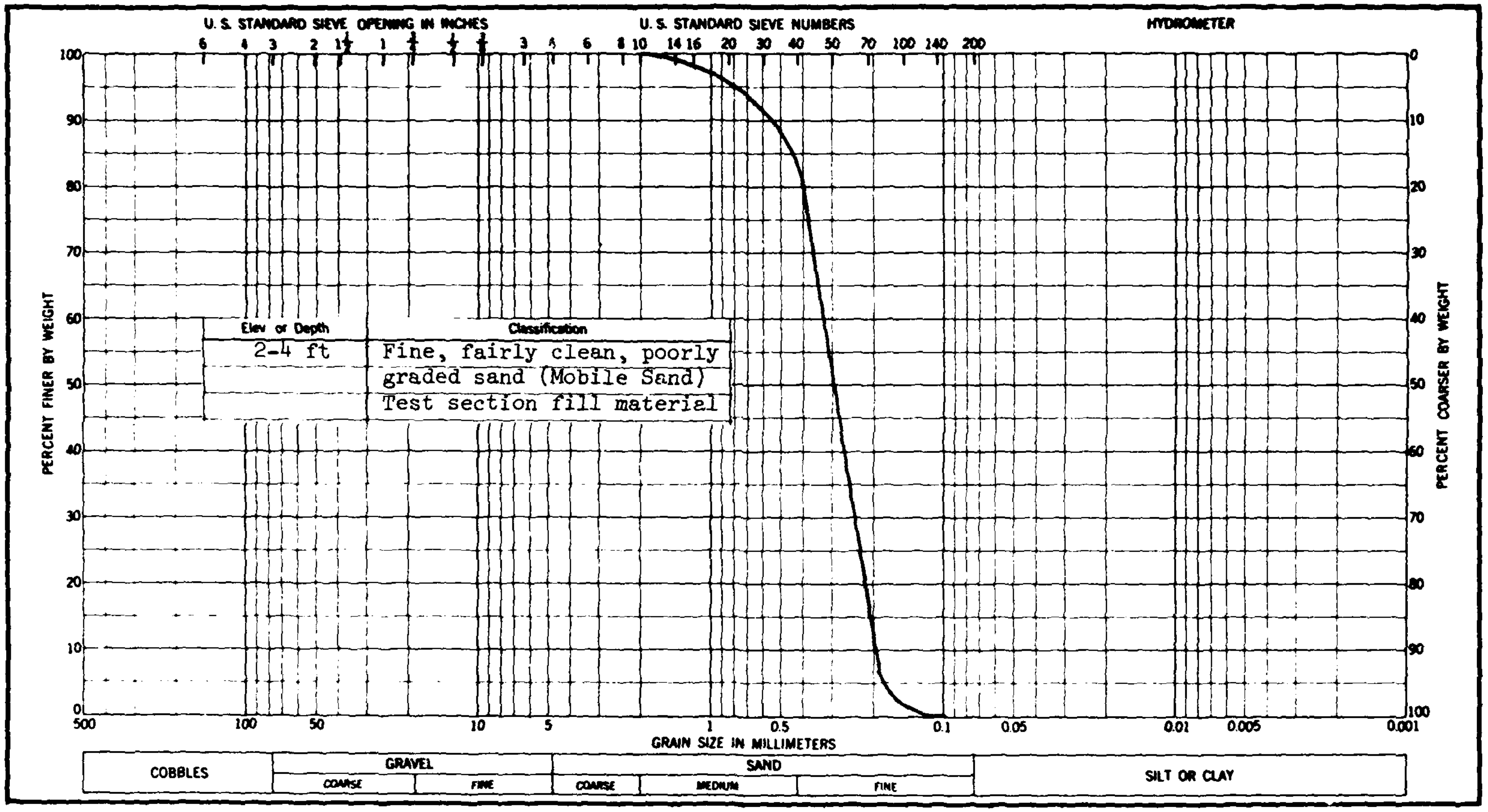



Figure 29. Gradation of Sand Used as Fill Material

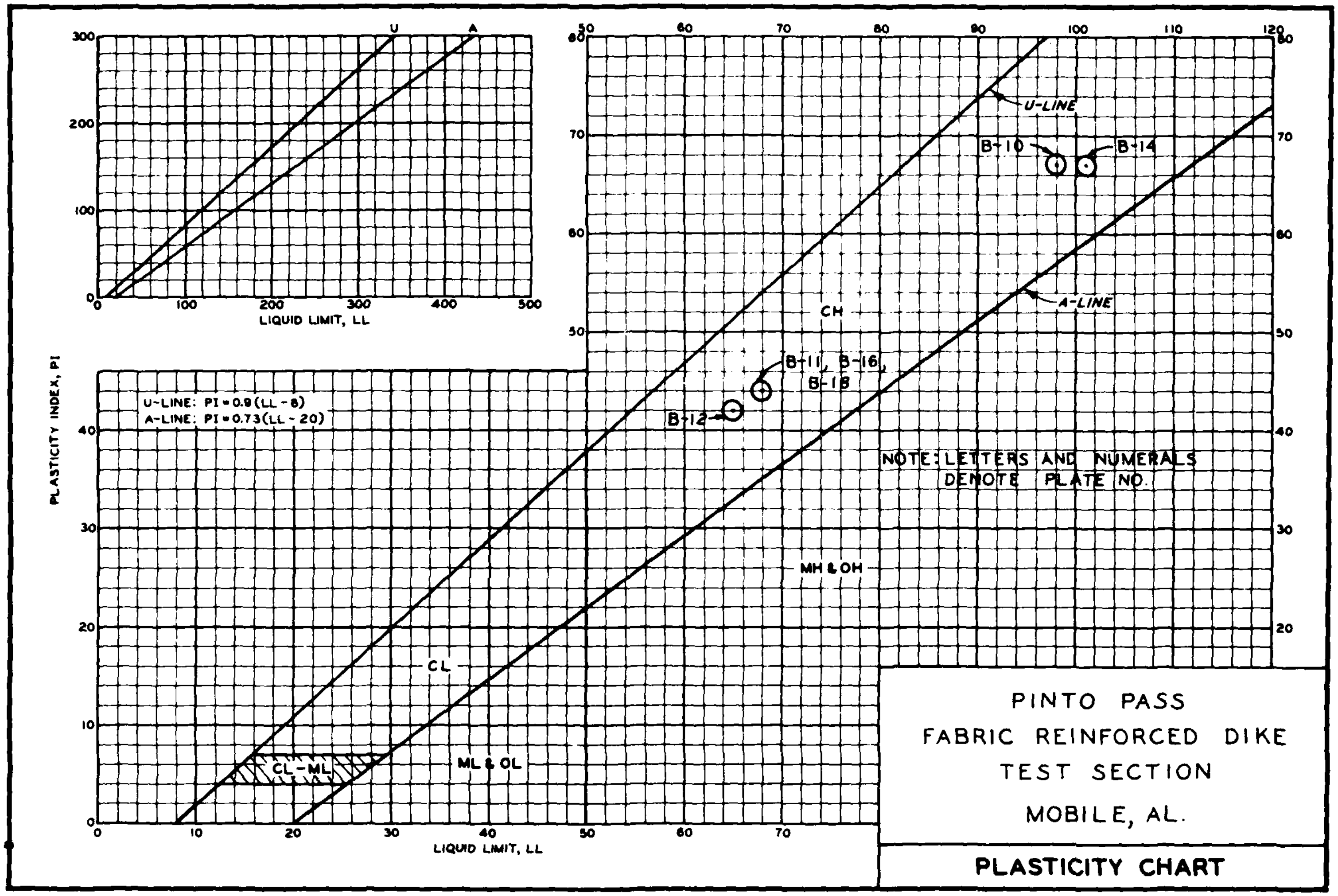
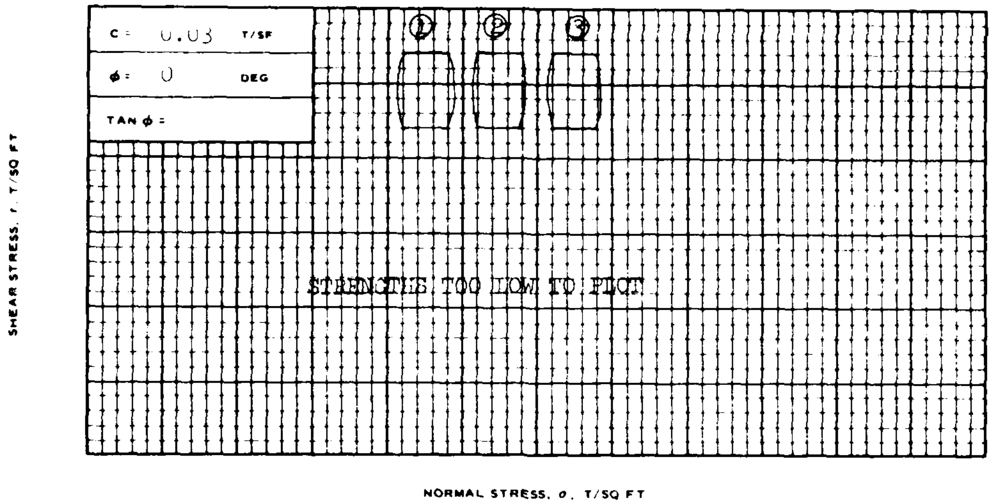
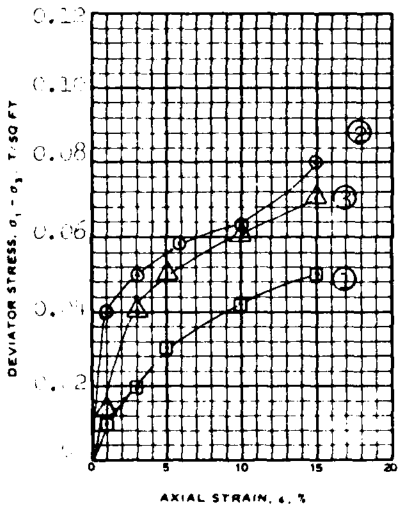



Figure 30. Plasticity Index versus Liquid Limit of Undisturbed Samples Used in Triaxial Tests




| | | | | 1 | 2 | 3 | |
|---|---|---|---|---|---|---|---|
| SPECIMEN NO. | | | | 1 | 2 | 3 | |
| INITIAL | WATER CONTENT, % | | $w_o$ | 165.9 | 117.8 | 106.4 | |
| | DRY DENSITY LB/CU FT | | $\gamma_{d_o}$ | 30.0 | 40.6 | 42.9 | |
| | SATURATION, % | | $S_o$ | 96.6 | 100+ | 97.5 | |
| | VOID RATIO | | $e_o$ | 4.72 | 3.23 | 3.00 | |
| BEFORE SHEAR | WATER CONTENT, % | | $w_c$ | | | | |
| | DRY DENSITY LB/CU FT | | $\gamma_{d_c}$ | | | | |
| | SATURATION, % | | $S_c$ | | | | |
| | VOID RATIO | | $e_c$ | | | | |
| | FINAL BACK PRESSURE, T/SQ FT | | $u_o$ | | | | |
| MINOR PRINCIPAL STRESS, T/SQ FT | | | $\sigma_3$ | 0.25 | 0.5 | 1.0 | |
| MAXIMUM DEVIATOR STRESS, T/SQ FT | | | $(\sigma_1 - \sigma_3)_{max}$ | 0.05 | 0.08 | 0.07 | |
| TIME TO $(\sigma_1 - \sigma_3)_{max}$, MIN | | | $t_f$ | 39 | 30 | 34 | |
| ULTIMATE DEVIATOR STRESS, T/SQ FT | | | $(\sigma_1 - \sigma_3)_{ult}$ | | | | |
| INITIAL DIAMETER, IN. | | | $D_o$ | 1.40 | 1.41 | 1.42 | |
| INITIAL HEIGHT, IN. | | | $H_o$ | 3.00 | 3.00 | 3.00 | |

CONTROLLED- strain TEST

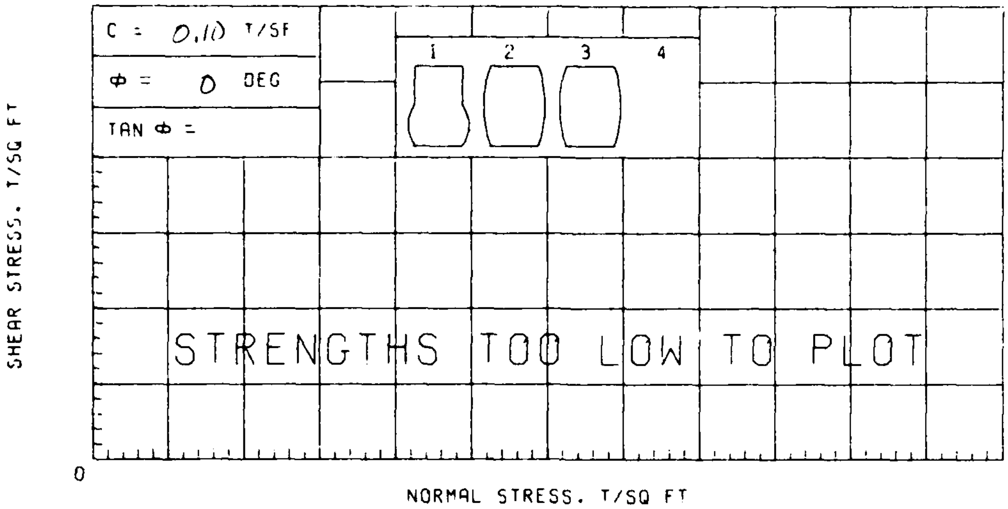
DESCRIPTION OF SPECIMENS PLASTIC CLAY(CH), brownish gray

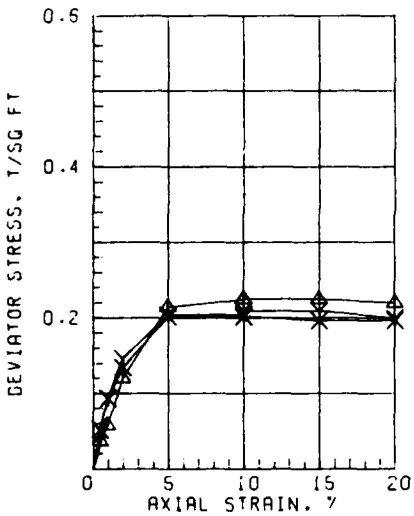
| | | | |
|---|---|---|---|
| LL 98 | PL 31 | PI 67 | Gs 2.75 |

| | |
|---|---|
| TYPE OF SPECIMEN UNDISTURBED | TYPE OF TEST Q |
| PROJECT PINTO PASS, MOBILE, AL. | |
| BORING NO | SAMPLE NO |
| DEPTH ELEV 5.0-5.5 | |
| LABORATORY USAEWES | DATE 05MAY77 |
| JMS | TRIAXIAL COMPRESSION TEST REPORT |

REMARKS Insufficient material for preparing 4th specimen.

Figure 31. Triaxial Compression Test Report on Q Test for Samples from 5.0-5.5 ft Depth




CONTROLLED-STRAIN TEST

| | SPECIMEN NO. | △1 | Y2 | X3 | 4 |
|---|---|---|---|---|---|
| INITIAL | WATER CONTENT, % | 71.3 | 67.9 | 87.7 | |
| | DRY DENSITY, PCF | 58.2 | 59.4 | 49.8 | |
| | SATURATION, % | 100+ | 99.3 | 98.9 | |
| | VOID RATIO | 1.916 | 1.861 | 2.412 | |
| BEFORE SHEAR | WATER CONTENT, % | | | | |
| | DRY DENSITY, PCF | | | | |
| | SATURATION, % | | | | |
| | VOID RATIO | | | | |
| | BACK PRESS., TSF | | | | |
| | MIN PRIN. STRESS, TSF | 0.3 | 0.5 | 1.0 | |
| | MAX. DEV. STRESS, TSF | 0.22 | 0.20 | 0.20 | |
| | TIME TO FAILURE, MIN. | 21 | 7 | 6 | |
| | RATE OF STRAIN INCR.% | | | | |
| | INITIAL DIAMETER, IN. | 1.37 | 1.37 | 1.38 | |
| | INITIAL HEIGHT, IN. | 3.00 | 3.00 | 3.00 | |

DESCRIPTION OF SPECIMENS: PLASTIC CLAY (CH), BLACKISH GRAY

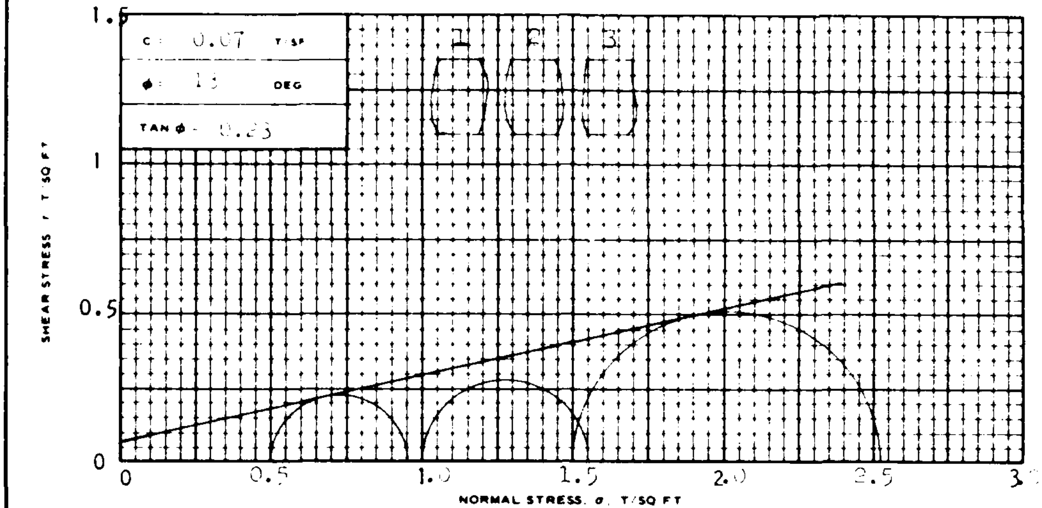
| | | | | | |
|---|---|---|---|---|---|
| LL 68 | PL 24 | PI 44 | GS 2.72 | UNDISTURBED SPECIMEN | Q TEST |

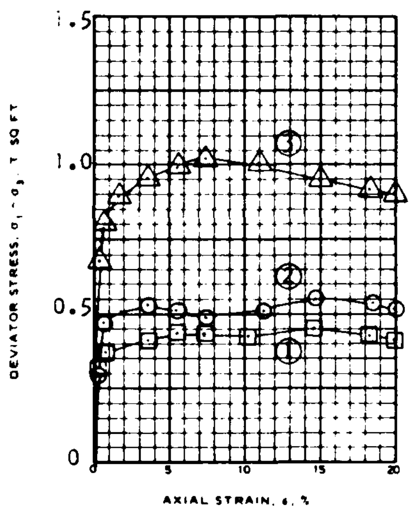
REMARKS:

| | |
|---|---|
| PROJECT PINTO PASS - EMBANKMENT TEST | |
| BORING NO. 6+00 | SAMPLE NO. 1 |
| DEPTH/ELEV 9.0-12.0 | TECH. JMS |
| LABORATORY USAE WES | DATE 06 FEB 79 |

TRIAXIAL COMPRESSION TEST REPORT

Figure 32. Triaxial Compresson Test Report on Q Test for Samples from 9.0-12.0 ft Depth




| SPECIMEN NO. | | | 1 | 2 | 3 |
|---|---|---|---|---|---|
| INITIAL | WATER CONTENT, % | $w_o$ | 121.8 | 93.1 | 92.3 |
| | DRY DENSITY LB/CU FT | $\gamma_{d_o}$ | 38.9 | 48.3 | 47.4 |
| | SATURATION, % | $S_o$ | 98.2 | 100+ | 96.8 |
| | VOID RATIO | $e_o$ | 3.41 | 2.56 | 2.62 |
| BEFORE SHEAR | WATER CONTENT, % | $w_c$ | 83.6 | 61.8 | 61.7 |
| | DRY DENSITY LB/CU FT | $\gamma_{d_c}$ | 47.0 | 58.3 | 60.4 |
| | SATURATION, % | $S_c$ | 86.7* | 87.5* | 92.2* |
| | VOID RATIO | $e_c$ | 2.65 | 1.94 | 1.84 |
| | FINAL BACK PRESSURE, T/SQ FT | $u_o$ | 2.88 | 2.88 | 2.88 |
| MINOR PRINCIPAL STRESS, T/SQ FT | | $\sigma_3$ | 0.5 | 1.0 | 1.5 |
| MAXIMUM DEVIATOR STRESS, T/SQ FT | | $(\sigma_1 - \sigma_3)_{max}$ | 0.45 | 0.55 | 1.02 |
| TIME TO $(\sigma_1 - \sigma_3)_{max}$, MIN | | $t_f$ | 1741 | 1765 | 8824 |
| ULTIMATE DEVIATOR STRESS, T/SQ FT | | $(\sigma_1 - \sigma_3)_{ult}$ | | | |
| INITIAL DIAMETER, IN. | | $D_o$ | 1.38 | 1.38 | 1.39 |
| INITIAL HEIGHT, IN. | | $H_o$ | 3.00 | 3.00 | 3.00 |

CONTROLLED- strain TEST

DESCRIPTION OF SPECIMENS PLASTIC CLAY (CH), brownish gray and black

LL 65 | PL 23 | PI 42 | G 2.74 | TYPE OF SPECIMEN UNDISTURBED | TYPE OF TEST R

REMARKS

See attached sheet for effective values.* Pore pressure response indicated 100% saturation. Insufficient material for preparing 4th specimen. (con't.)

PROJECT PINTO PASS, MOBILE, AL.

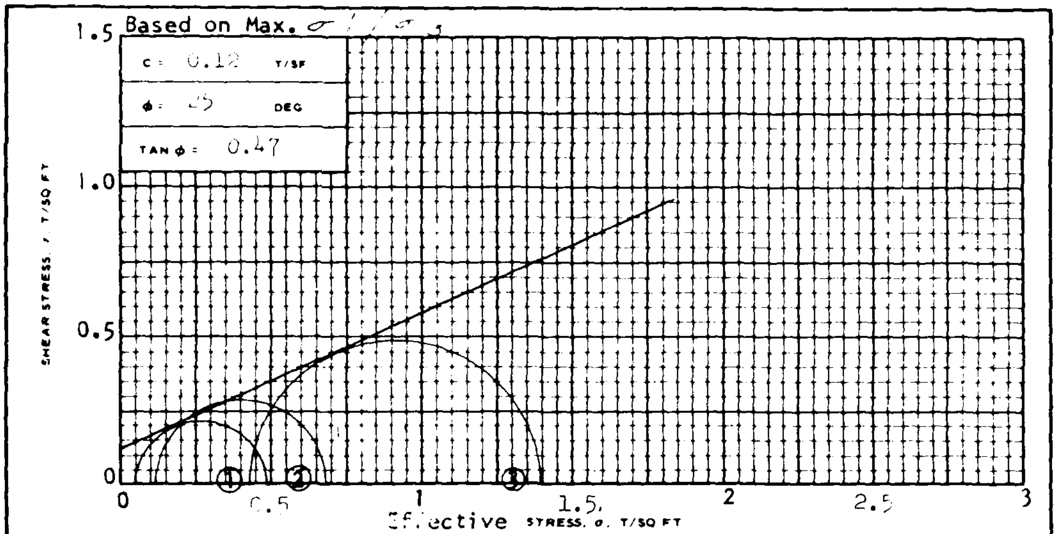
BORING NO | SAMPLE NO

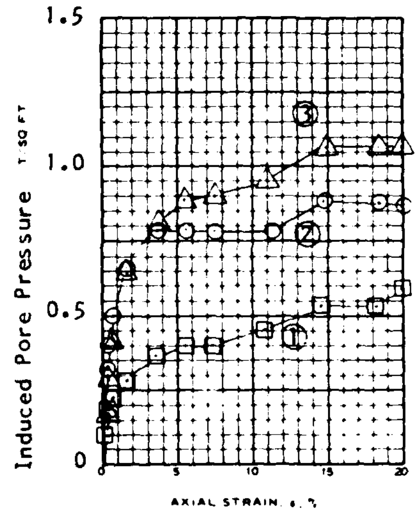
DEPTH/ELEV 05.0 to -5.5 ft MSL

LABORATORY USAEWES | DATE 9 MAY 77

JMS TRIAXIAL COMPRESSION TEST REPORT

Figure 33. Triaxial Compression Test Report on R Test for Samples from 5.0-5.5 ft Depth




| SPECIMEN NO. | | | | | | |
|---|---|---|---|---|---|---|
| INITIAL | WATER CONTENT, % | $w_o$ | | | | |
| | DRY DENSITY LB/CU FT | $\gamma_{d_o}$ | | | | |
| | SATURATION, % | $S_o$ | | | | |
| | VOID RATIO | $e_o$ | | | | |
| BEFORE SHEAR | WATER CONTENT, % | $w_c$ | | | | |
| | DRY DENSITY LB/CU FT | $\gamma_{d_c}$ | | | | |
| | SATURATION, % | $S_c$ | | | | |
| | VOID RATIO | $e_c$ | | | | |
| | FINAL BACK PRESSURE T/SQ FT | $u_o$ | | | | |
| MINOR PRINCIPAL STRESS, T/SQ FT | | $\sigma_3$ | | | | |
| MAXIMUM DEVIATOR STRESS, T/SQ FT | | $(\sigma_1 - \sigma_3)_{max}$ | | | | |
| TIME TO $(\sigma_1 - \sigma_3)_{max}$, MIN | | $t_f$ | | | | |
| ULTIMATE DEVIATOR STRESS, T/SQ FT | | $(\sigma_1 - \sigma_3)_{ult}$ | | | | |
| INITIAL DIAMETER, IN. | | $D_o$ | | | | |
| INITIAL HEIGHT, IN | | $H_o$ | | | | |

CONTROLLED- strain TEST

DESCRIPTION OF SPECIMENS

| LL | PL | PI | Gs | TYPE OF SPECIMEN | TYPE OF TEST $\overline{R}$ |
|---|---|---|---|---|---|

REMARKS SAMPLES ALLOWED TO AIR DRY FOR APPROX. 2 HOURS BEFORE TRIMMING TEST SPECIMENS.

PROJECT PINTO PASS, MOBILE, ALA.

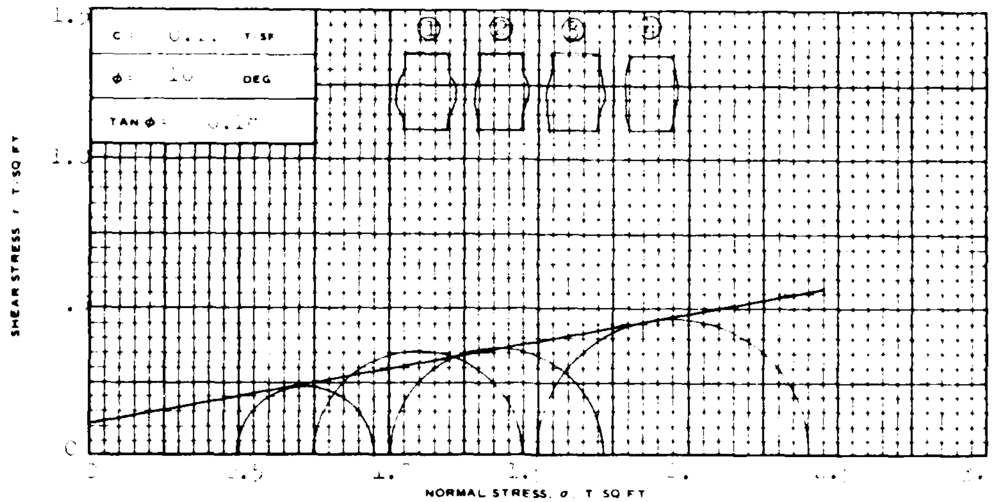
BORING NO | SAMPLE NO.

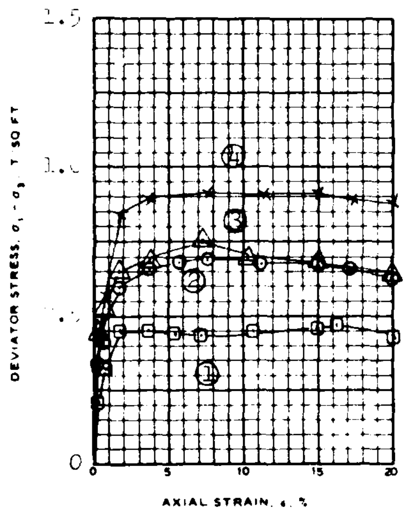
DEPTH ELEV -5.0 to -5.5 ft MSL

LABORATORY USAEWES | DATE 9 MAY 1977

JMS TRIAXIAL COMPRESSION TEST REPORT

Figure 34. Triaxial Compression Test Report of $\overline{R}$ Test for Samples from 5.0-5.5 ft Depth




| SPECIMEN NO | | | 1 | 2 | 3 | [illegible] |
|---|---|---|---|---|---|---|
| INITIAL | WATER CONTENT, % | $w_o$ | 108.3 | 111.5 | 86.1 | [illegible] |
| | DRY DENSITY LB/CU FT | $\gamma d_o$ | 42.3 | [illegible]1.7 | [illegible] | [illegible]2.2 |
| | SATURATION, % | $S_o$ | 97.7 | 98.6 | 98.1 | [illegible] |
| | VOID RATIO | $e_o$ | 3.03 | [illegible] | [illegible] | 3.03 |
| BEFORE SHEAR | WATER CONTENT, % | $w_c$ | 77.8 | 69.5 | 56.5 | [illegible] |
| | DRY DENSITY LB/CU FT | $\gamma d_c$ | 49.8 | 56.5 | 66.7 | [illegible] |
| | SATURATION, % | $S_c$ | 87.7* | 94.2* | 99.2 | [illegible]9.3 |
| | VOID RATIO | $e_c$ | 2.42 | 2.02 | 1.55 | 1.85 |
| | FINAL BACK PRESSURE, T/SQ FT | $u_o$ | 2.88 | 2.88 | 2.88 | 2.88 |
| MINOR PRINCIPAL STRESS, T/SQ FT | | $\sigma_3$ | 0.5 | 0.75 | 1.0 | 1.5 |
| MAXIMUM DEVIATOR STRESS, T/SQ FT | | $(\sigma_1 - \sigma_3)_{max}$ | 0.45 | 0.69 | 0.72 | 0.91 |
| TIME TO $(\sigma_1 - \sigma_3)_{max}$, MIN | | $t_f$ | 225 | 7575 | 7475 | 856 |
| ULTIMATE DEVIATOR STRESS, T/SQ FT | | $(\sigma_1 - \sigma_3)_{ult}$ | | | | |
| INITIAL DIAMETER, IN. | | $D_o$ | 1.41 | 1.41 | 1.40 | 1.41 |
| INITIAL HEIGHT, IN | | $H_o$ | 3.00 | 3.00 | 3.00 | 3.00 |

CONTROLLED- STRAIN TEST

DESCRIPTION OF SPECIMENS PLASTIC CLAY(CH), brownish gray and black

LL 101 | PL 34 | PI 67 | Gs 2.73 | TYPE OF SPECIMEN UNDISTURBED | TYPE OF TEST R

REMARKS SEE ATTACHED SHEET FOR EFFECTIVE VALUES. *PORE PRESSURE RESPONSE INDICATED 100% SATURATION. SAMPLES ALLOWED TO AIR DRY FOR 2 HOURS BEFORE TRIMMING SPECIMENS.

PROJECT PINTO PASS, MOBILE, AL.

BORING NO

SAMPLE NO

DEPTH ELEV -5.0 to -5.5 ft MSL

LABORATORY USAEWES

DATE 16MAY77

JMS

TRIAXIAL COMPRESSION TEST REPORT

Figure 35. Triaxial Compression Test Report on $R_1$ Test for Samples from 5.0-5.5 ft Depth

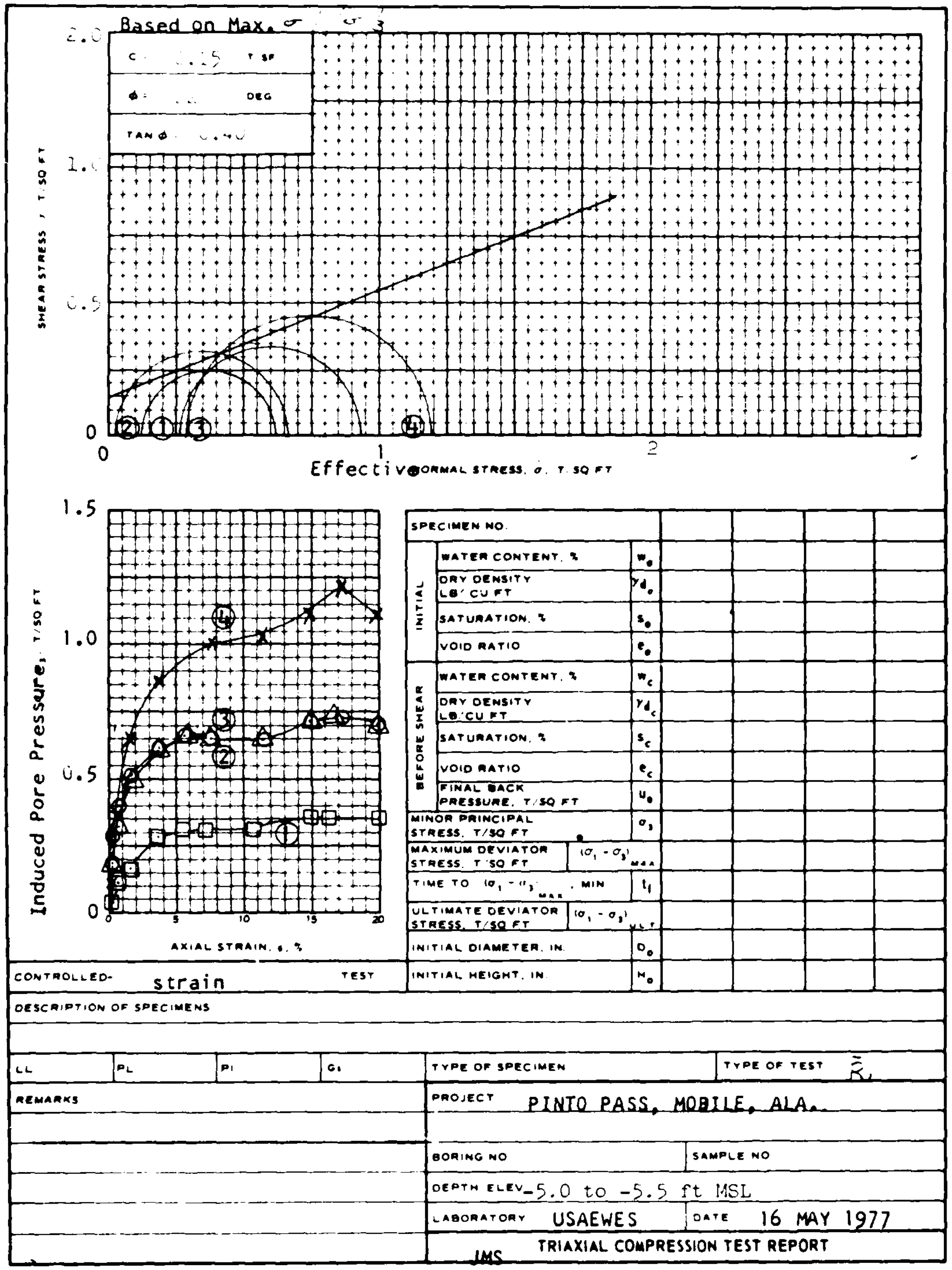
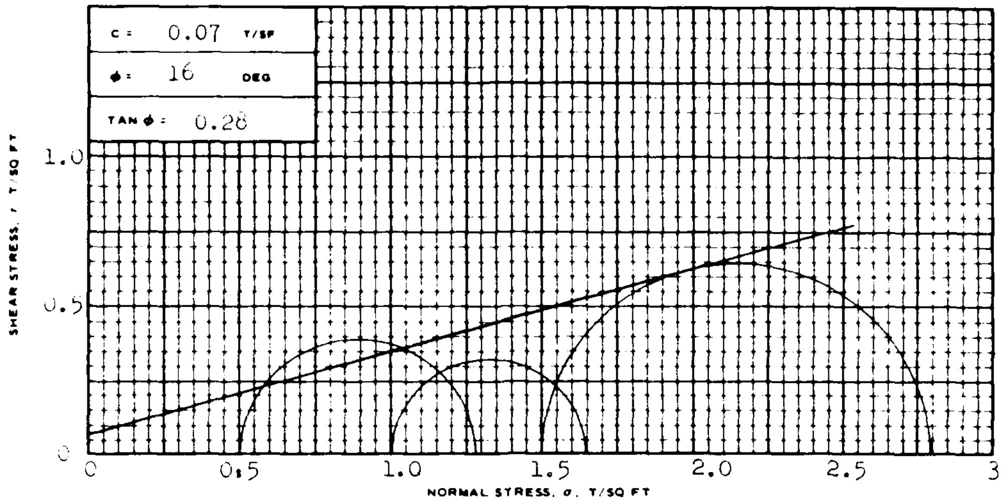
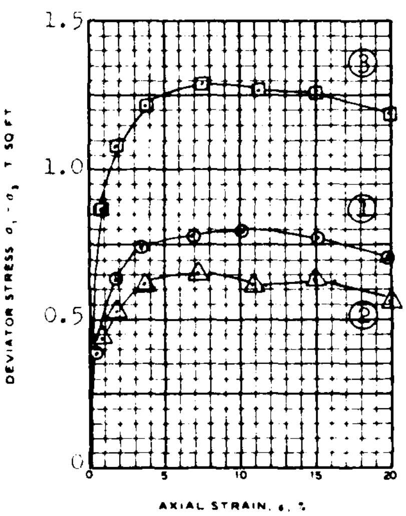



Figure 36. Triaxial Compression Test Report on $\bar{R}_1$ Test for Samples from 5.0-5.5 ft Depth



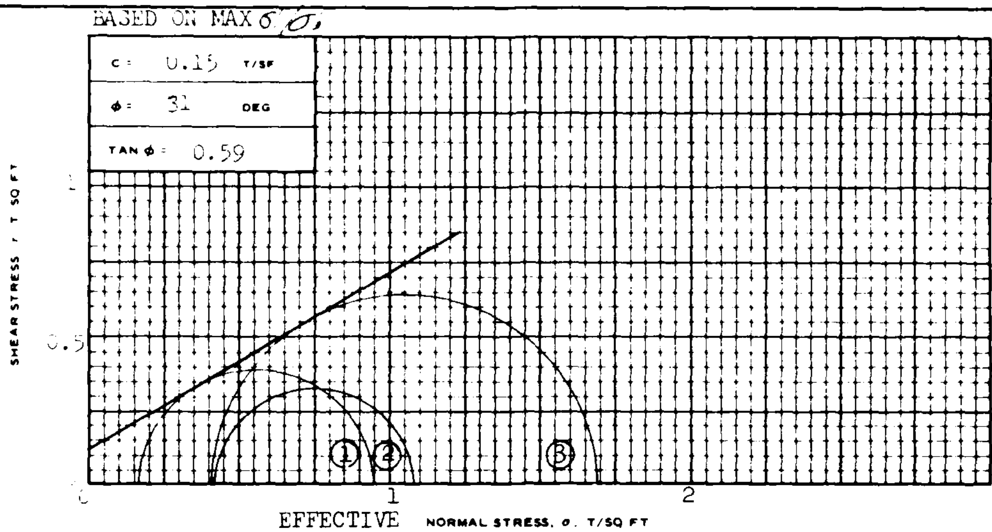

| SPECIMEN NO. | | | 1 | 2 | 3 | |
|---|---|---|---|---|---|---|
| INITIAL | WATER CONTENT, % | $w_o$ | 46.3 | 72.3 | 81.5 | |
| | DRY DENSITY LB/CU FT | $\gamma_{d_o}$ | 74.6 | 57.3 | 53.[illegible] | |
| | SATURATION, % | $S_o$ | 98.7 | 100+ | 100+ | |
| | VOID RATIO | $e_o$ | 1.276 | 1.963 | 2.204 | |
| BEFORE SHEAR | WATER CONTENT, % | $w_c$ | 41.3 | 53.7 | 55.2 | |
| | DRY DENSITY LB/CU FT | $\gamma_{d_c}$ | 79.4 | 69.6 | 68.8 | |
| | SATURATION, % | $S_c$ | 98.6* | 100+ | 100+ | |
| | VOID RATIO | $e_c$ | 1.139 | 1.441 | 1.468 | |
| | FINAL BACK PRESSURE, T/SQ FT | $u_o$ | 3.6 | 3.6 | 3.6 | |
| MINOR PRINCIPAL STRESS, T/SQ FT | | $\sigma_3$ | 0.5 | 1.0 | 1.5 | |
| MAXIMUM DEVIATOR STRESS, T/SQ FT | | $(\sigma_1 - \sigma_3)_{max}$ | 0.77 | 0.64 | 1.27 | |
| TIME TO $(\sigma_1 - \sigma_3)_{max}$, MIN | | $t_f$ | 1071 | 1071 | 1071 | |
| ULTIMATE DEVIATOR STRESS, T/SQ FT | | $(\sigma_1 - \sigma_3)_{ult}$ | | | | |
| INITIAL DIAMETER, IN | | $D_o$ | 1.39 | 1.39 | 1.39 | |
| INITIAL HEIGHT, IN | | $H_o$ | 3.00 | 3.00 | 3.00 | |

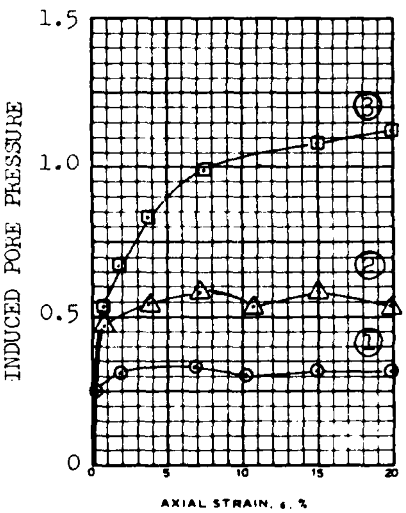
CONTROLLED- STRAIN TEST

DESCRIPTION OF SPECIMENS PLASTIC CLAY (CH), BLACKISH GRAY; ORGANIC MATTER; SAND LENSES

| | | | | | |
|---|---|---|---|---|---|
| LL 68 | PL 24 | PI 44 | Gs 2.72 | TYPE OF SPECIMEN Undisturbed | TYPE OF TEST R |
| REMARKS *PORE PRESSURE RESPONSE INDICATED 100% SATURATION | | | | PROJECT PINTO PASS - EMBANKMENT TEST | |
| | | | | BORING NO 7+00 | SAMPLE NO |
| | | | | DEPTH ELEV -10.5 to -12.0 ft MSL | |
| | | | | LABORATORY USAE WES | DATE 17 FEB 79 |
| | | | | JMS TRIAXIAL COMPRESSION TEST REPORT | |

Figure 37. Triaxial Compression Test Report on R Test for Samples from 10.5-12.0 ft Depth




| | | | | | | |
|---|---|---|---|---|---|---|
| SPECIMEN NO. | | | | | | |
| INITIAL | WATER CONTENT, % | $w_o$ | | | | |
| | DRY DENSITY LB/CU FT | $\gamma d_o$ | | | | |
| | SATURATION, % | $S_o$ | | | | |
| | VOID RATIO | $e_o$ | | | | |
| BEFORE SHEAR | WATER CONTENT, % | $w_c$ | | | | |
| | DRY DENSITY LB/CU FT | $\gamma d_c$ | | | | |
| | SATURATION, % | $S_c$ | | | | |
| | VOID RATIO | $e_c$ | | | | |
| | FINAL BACK PRESSURE, T/SQ FT | $u_o$ | | | | |
| MINOR PRINCIPAL STRESS, T/SQ FT | | $\sigma_3$ | | | | |
| MAXIMUM DEVIATOR STRESS, T/SQ FT | | $(\sigma_1 - \sigma_3)_{MAX}$ | | | | |
| TIME TO $(\sigma_1 - \sigma_3)_{MAX}$, MIN | | $t_f$ | | | | |
| ULTIMATE DEVIATOR STRESS, T/SQ FT | | $(\sigma_1 - \sigma_3)_{ULT}$ | | | | |
| INITIAL DIAMETER, IN. | | $D_o$ | | | | |
| INITIAL HEIGHT, IN. | | $H_o$ | | | | |

CONTROLLED- TEST

DESCRIPTION OF SPECIMENS

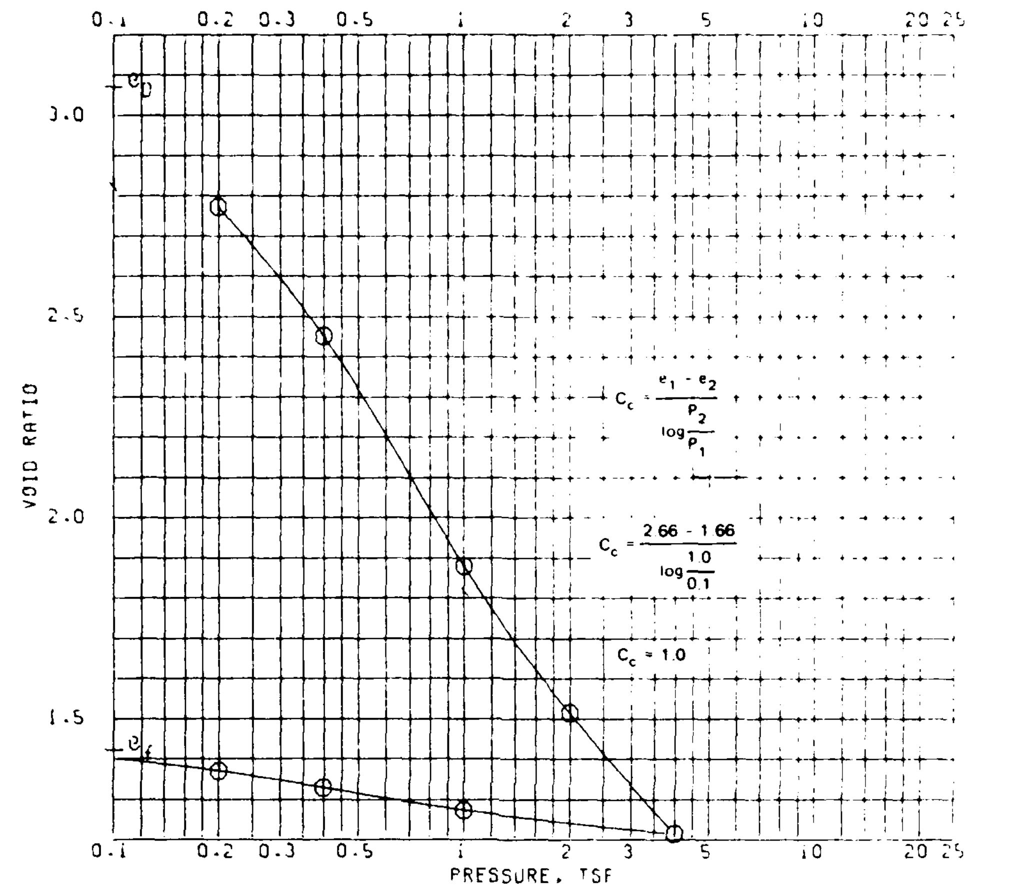
| LL | PL | PI | Gs | TYPE OF SPECIMEN | TYPE OF TEST $\bar{R}$ |
|---|---|---|---|---|---|

REMARKS

| | |
|---|---|
| PROJECT | PINTO PASS - EMBANKMENT TEST |
| BORING NO. 7+00 | SAMPLE NO. |
| DEPTH/ELEV | -10.5 to -12.0 ft MSL |
| LABORATORY USAEWES | DATE 17 FEB 79 |

**TRIAXIAL COMPRESSION TEST REPORT**

Figure 38. Triaxial Compression Test Report on $\bar{R}$ Test for Samples from 10.5-12.0 ft Depth




| | | | BEFORE TEST | AFTER TEST |
|---|---|---|---|---|
| OVERBURDEN PRESSURE, TSF | | WATER CONTENT, % | 111.0 | 54.7 |
| PRECONSOL. PRESSURE, TSF | | DRY DENSITY, PCF | 41.7 | 70.2 |
| COMPRESSION INDEX | | SATURATION, % | 98.4 | 100 + |
| TYPE SPECIMEN | UNDISTURBED | VOID RATIO | 3.057 | 1.413 |
| DIA. IN 2.50 | HT. IN 1.505 | BACK PRESSURE, TSF | | |
| CLASSIFICATION PLASTIC CLAY (CH), BROWNISH GRAY | | | | |
| LL 68 PL 24 PI 44 | | PROJECT PINTO PASS, EMBANKMENT STUDY | | |
| GS 2.72 | $D_{10}$ | STATION 7+00 | | |
| REMARKS | | BORING NO. | SAMPLE NO. | |
| | | DEPTH/ELEV 9.0-12.0 | DATE 21 FEB 79 | |
| | | CONSOLIDATION TEST REPORT | | |

**Figure 39. Consolidation Test Report for Samples from 9.0-12.0 ft Depth**

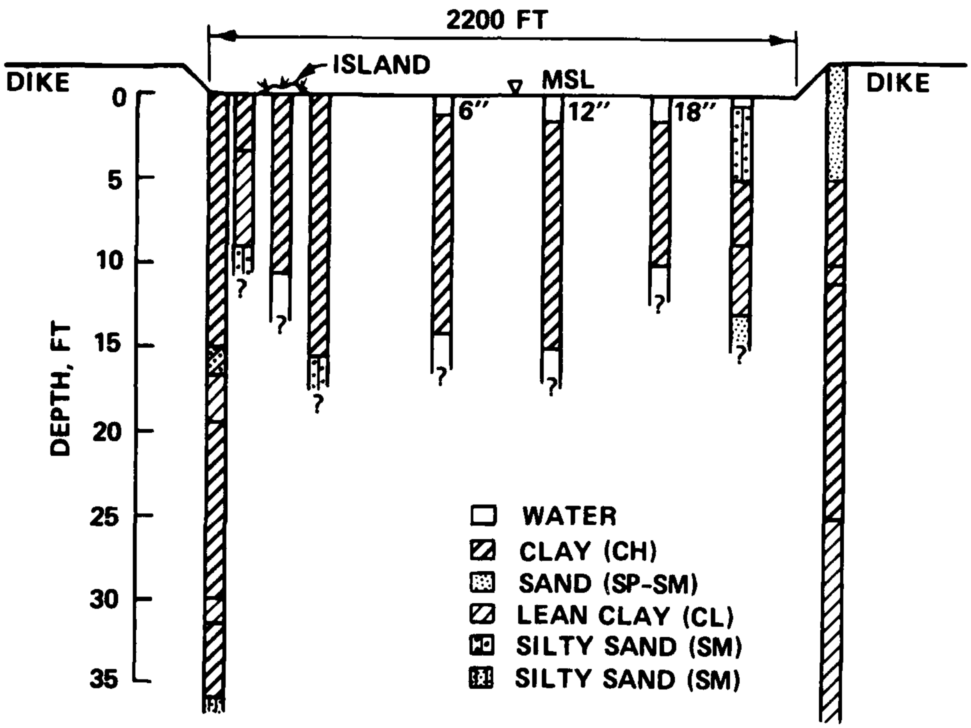



Figure 40. Soil Profile at East End of Pinto Pass

# APPENDIX B

## PHOTOGRAPHS OF CONSTRUCTION SEQUENCE

This appendix is included to illustrate photographically the construction sequence of the embankment test section at Pinto Pass.

Figure 41. Dragline Loading Dump Truck at Dredged Material Disposal Area Borrow Pit

Figure 42. John Deere 350 Wide-Track (28 in.) Dozer Spreading a 1-ft Layer of Borrow Material onto Grass Covering

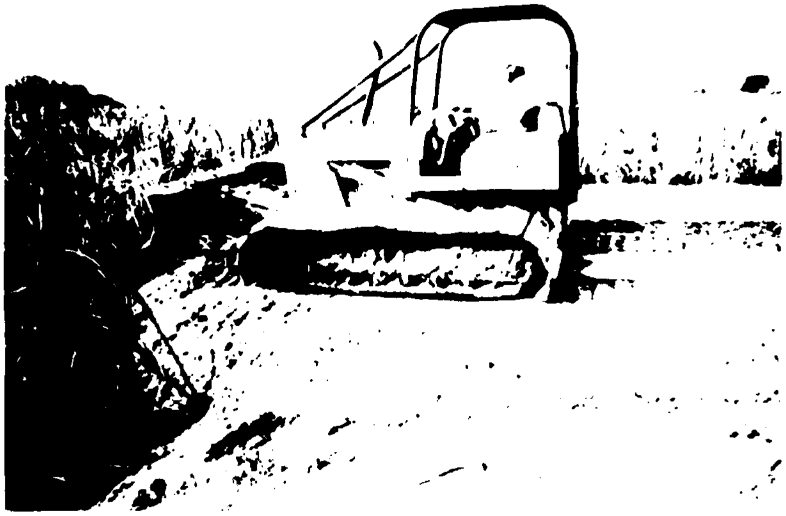


Figure 43. Wide-Track Dozer Spreading a 1-ft Layer of Borrow Material onto Cattail Vegetation Cover

Figure 44. Dozer Spreading Dry Sand. Note Right Portion of Photo Shows Pore Water that Had Risen Overnight through Sand and Was Running off of the Working Table

Figure 45. Advance Type I Fabric Being Placed, with Seams Perpendicular to Longitudinal Axis of Embankment, onto Sand Working Table

Figure 46. Placement of Advance Type I Fabric Prior to Placement of Fill Material

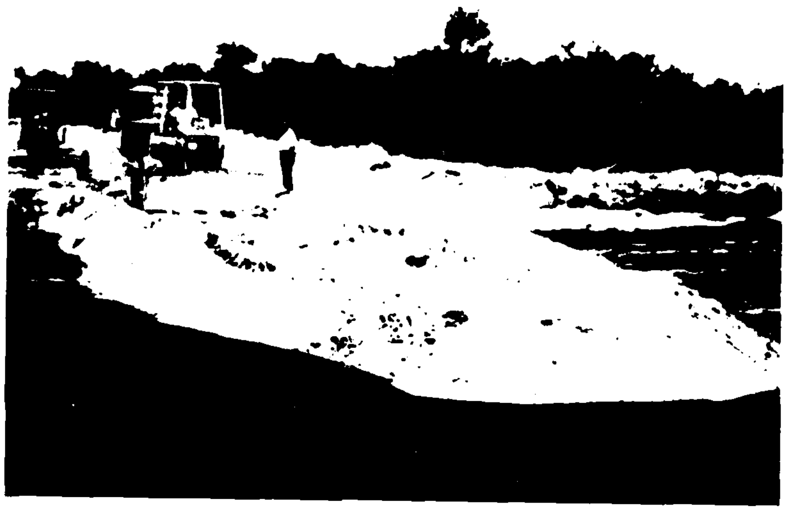


Figure 47. Construction of Parallel Haul Road at Toe of Embankment

Figure 48. Fabric Folded Back into Toe of Structure to Serve as an Anchor to Prevent Fabric from Slipping

Figure 49. Dozer Advancing Sand Working Table Toward Pinto Pass Channel

Figure 50. Mud Waves Breaking through Sand Working Table from too Many Passes by Dozer in Spreading Material

Figure 51. Termination of Sand Working Table at Edge of Pinto Pass Channel

Figure 52. Polyfilter X Fabric Being Rolled onto Advancing Mud Wave Caused by Fill Material and Fabric Displacement of Soft Foundation Materials

Figure 53. Aerial View of Fabric-Reinforced Embankment Being Constructed Across Pinto Pass

Figure 54. Nicolon 66475 Fabric after Placement and Folding Back for Sewing on Advancing Mud Wave

Figure 55. Nicolon 66475 Fabric Edge Being Sewn Together Prior to Folding the Newly Sewn Fabric Forward

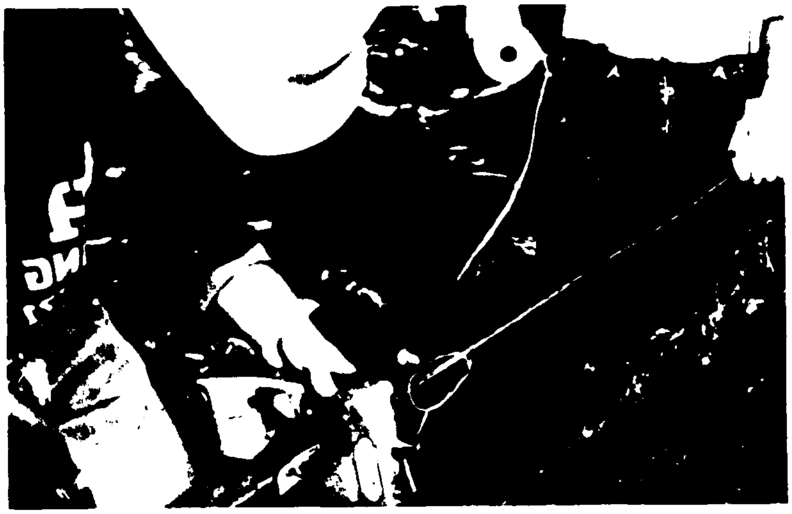


Figure 56. Close-up of Nicolon 66475 Fabric Being Sewn with a Sac-Up Model BB Hand-Held Field Sewing Machine

Figure 57 Nicolon 66475 and Nicolon 66186 After Being Sewn Together

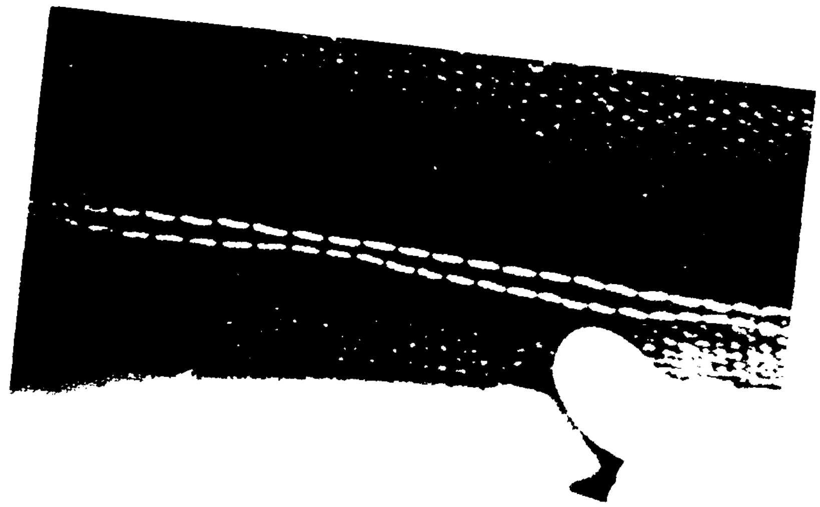


Figure 58. Top Side of Chain Stitch Joining Nicolon 66475 and Nicolon 66186 with 3 to 5 Stitches Per Inch

Figure 59. View Showing the Bottom Side of Chain Stitch

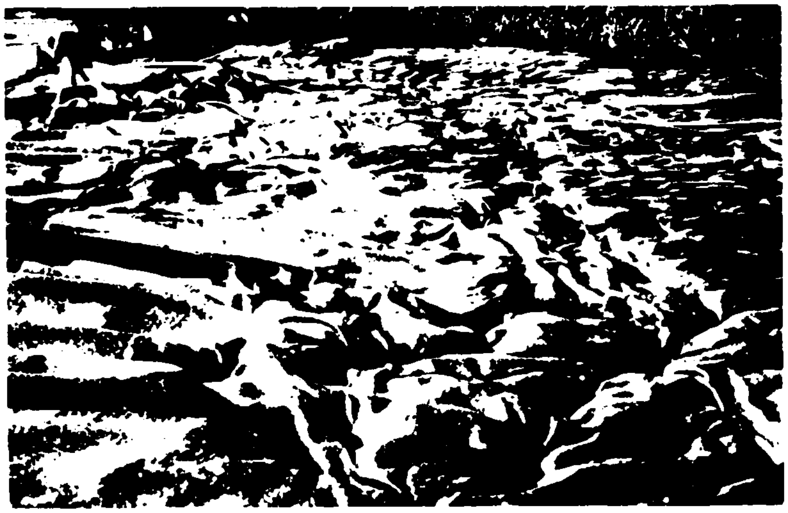


Figure 60. Footprints in Fabric Laid onto Advancing Mud Wave

Figure 61, Fabric Being Stretched by Placement of Fill Material on Advancing Mud Wave Beneath the Fabric

Figure 62. Nicolon 66475 Fabric, Placed Over Soft Foundation Material, Supporting Personnel

Figure 63. Fabric Seam Field-Sewn with Improper Thread Caused Failure to Occur Before Thread Was Replaced with Proper Type

Figure 64. Survey Crew Laying Out Piezometer and Settlement Locations

Figure 65. Settlement Plate Being Installed on Fabric

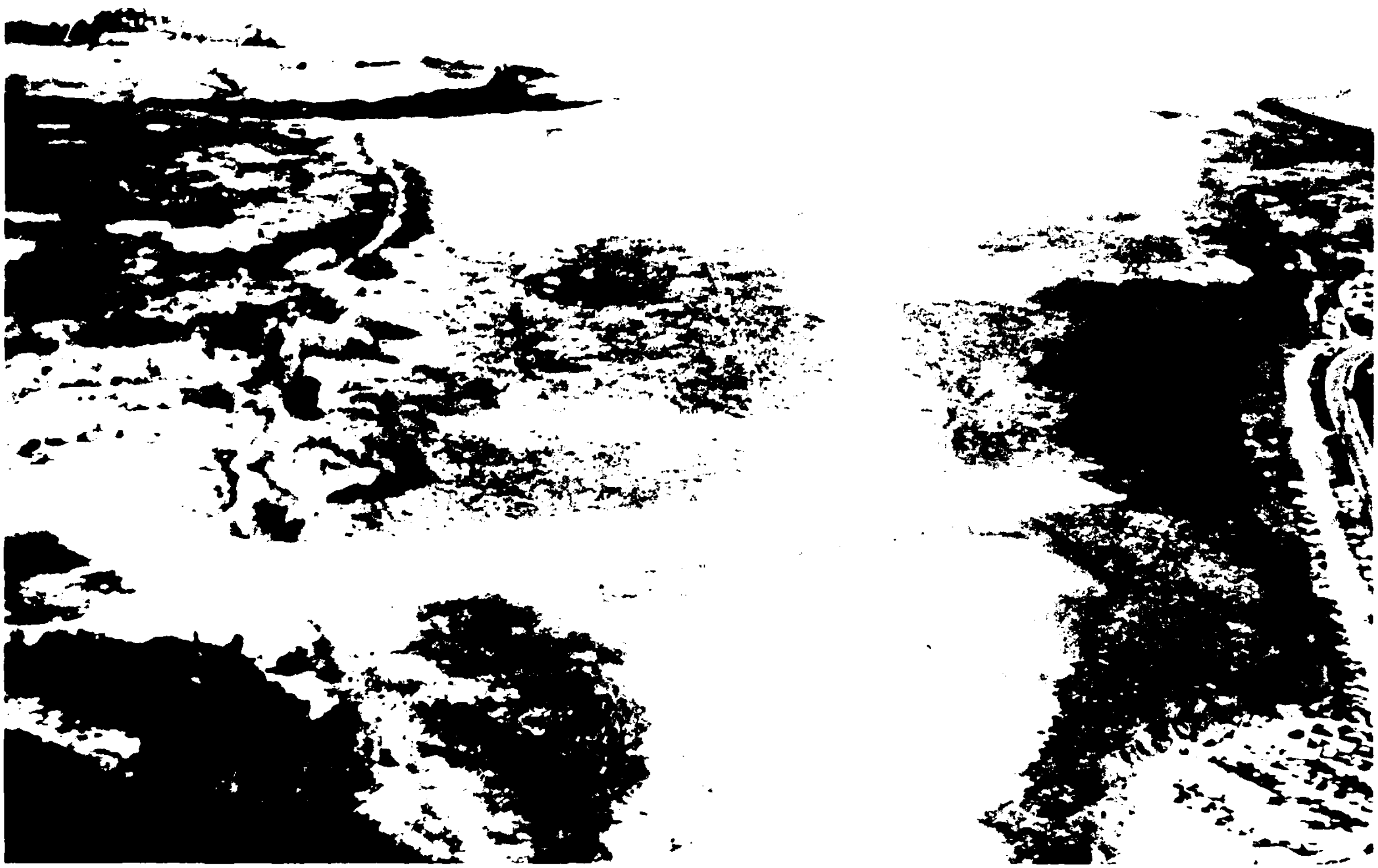


Figure 66. Fabric-Reinforced Embankment Test Section Completed Across Pinto Pass

# APPENDIX C

## SETTLEMENT AND PIEZOMETER INSTALLATION AND DATA

### Introduction

The field instrumentation used in the Pinto Pass test section embankment consisted of the most simple and reliable devices known for measuring embankment movement and hydrostatic pressure in a soil mass. Settlement plates were installed on the fabric to monitor movement in both horizontal and vertical directions, and piezometers were selected to measure the hydrostatic pressure beneath the embankment. These instruments were installed by MDO drill crew under the supervision of WES personnel.

### Settlement Plates

Thirty-five 18-in.-square plates, 3/4 in. thick, with a 3/4-in.-diam steel pipe risers extending above the embankment fill were installed along the longitudinal axis of the test section. Five settlement plates were installed at seven 100-ft stations on a line transverse to longitudinal axis of the embankment with one plate at each toe, one on the center line, and one between the toe and center line as shown in Figure 67. Immediately after the fabric was placed and after about one foot of backfill material was spread over the fabric, the soil was excavated and the settlement plates were installed directly onto the

fabric and a survey of the vertical and horizontal positions recorded. Figure 65, Appendix B, shows a typical settlement plate being installed.

Temporary bench marks used during layout and construction of the embankment to maintain control and monitor the vertical and horizontal movements of the embankment were replaced with permanent monuments near the end of construction. Once the plates and riser pipes were installed, settlement readings were recorded and plotted daily. These data are tabulated in Table XIV and plotted versus time in Figures 68 through 74, and settlement profiles for each station are plotted in Figures 75 through 78. Figure 79 shows a plan view of the test section and a plot of the maximum horizontal movement recorded at each settlement plate. The settlement plate risers were flagged and protected with three wooden stakes driven around the riser pipe to prevent dump trucks and dozers from inadvertently damaging the pipe and plate.

Piezometer

Fifty-six piezometers were installed in clusters of four at two locations each at seven transverse stations spaced 100 ft apart. Each location consisted of four piezometers located on the corner of a four-foot square with the piezometer tips located at about el -5, -10, -20, and -30. As there were no soil borings of the west end of Pinto Pass before piezometer installation, the instruments were located according to the location of the deep soft clay deposits found at the east end of Pinto Pass. A plan view showing the location of the piezometers is shown in Figure 67.

Porous heavy duty polyethylene piezometers, 24 in. long, purchased from Piezometer Research and Development Corporation, 33 Magee Avenue,

Stanford, CT, were installed in the soil beneath the Pinto Pass embankment test section. PVC adapters were provided to connect a 5/8-in. outside diameter PVC pipe to the piezometer that extended above the embankment surface. The piezometer PVC pipes along the embankment center line were initially cut off at el 9 MSL and the PVC pipes along the embankment toe were cut off at about el 5 MSL. Prior to installation, the piezometers were incased in woven polypropylene bags filled with a clean, coarse-grained concrete sand. Each piezometer hole was drilled either with clear water or revert drilling fluid to keep the holes open prior to installation of the piezometer. After each piezometer was installed, the 5/8-in. PVC pipe was filled with clean water and allowed to stand for 24 hours before readings were taken. Each pipe was covered with a pipe cap with a 1/8-in. hole drilled in the cap. Some of the piezometers had to be extended above el 9 MSL because, as construction progressed and the embankment height increased, the pore water pressure increased beyond the height of the cut-off, above el 11.2 MSL. All the piezometer readings were obtained with a M-scope that had an electrode on the end of a cable that was dropped down the inside of the PVC pipe to determine the water level. These readings were recorded daily and are tabulated in Table XIV and plotted as pore water in feet versus time in Figures 80 through 93.

TABLE XIV

SETTLEMENT READINGS, PINTO PASS TEST SECTION

| Date | Station | Undisturbed Ground el, ft | E0+58 S-1-1 | E0+36 S-1-2 | E0+06 S-1-3 | W0+36 S-1-4 | W0+58 S-1-5 |
|---|---|---|---|---|---|---|---|
| | | | Settlement Readings, MSL, ft | | | | |
| 11-16-78 | 1+00 | 1.4 | 2.71 | 2.52 | 2.49 | 2.48 | 2.88 |
| 11-17-78 | | | 2.70 | 2.52 | 2.54 | 2.47 | 2.87 |
| 11-20-78 | | | 2.71 | 2.42 | 2.50 | 2.37 | 2.88 |
| 11-29-78 | | | 2.68 | 2.39 | 2.39 | 2.36 | 2.79 |
| 11-30-78 | | | 2.67 | 2.40 | 2.44 | 2.36 | 2.80 |
| 12-01-78 | | | 2.67 | 2.39 | 2.40 | 2.36 | 2.79 |
| 12-04-78 | | | 2.68 | 2.41 | 2.46 | 2.38 | 2.82 |
| 12-05-78 | | | 2.67 | 2.39 | 2.40 | 2.36 | 2.79 |
| 12-06-78 | | | 2.66 | 2.38 | 2.38 | 2.34 | 2.78 |
| 12-07-78 | | | 2.68 | 2.39 | 2.39 | 2.35 | 2.79 |
| 12-08-78 | | | 2.67 | 2.37 | 2.37 | 2.33 | 2.77 |
| 12-11-78 | | | 2.67 | 2.38 | 2.36 | 2.34 | 2.77 |
| 12-12-78 | | | 2.67 | 2.37 | 2.34 | 2.33 | 2.76 |
| 12-13-78 | | | 2.67 | 2.36 | 2.32 | 2.33 | 2.76 |
| 12-14-78 | | | 2.62 | 2.35 | 2.32 | 2.33 | 2.77 |
| 12-15-78 | | | 2.67 | 2.36 | 2.31 | 2.32 | 2.76 |
| 12-17-78 | | | 2.67 | 2.33 | 2.28 | 2.30 | 2.72 |
| 12-18-78 | | | 2.64 | 2.32 | 2.26 | 2.29 | 2.72 |
| 12-20-78 | | | 2.62 | 2.21 | 2.22 | 2.28 | 2.70 |
| 12-21-78 | | | 2.65 | 2.33 | 2.28 | 2.29 | 2.73 |
| 12-22-78 | | | 2.64 | 2.32 | 2.27 | 2.29 | 2.72 |
| 12-26-78 | | | 2.66 | 2.31 | 2.24 | 2.25 | 2.71 |
| 12-28-78 | | | 2.64 | 2.32 | 2.24 | 2.29 | 2.72 |
| 12-30-78 | | | 2.65 | 2.31 | 2.23 | 2.27 | 2.71 |
| 12-31-78 | | | 2.65 | 2.31 | 2.22 | 2.28 | 2.72 |
| 01-02-79 | | | 2.63 | 2.31 | 2.21 | 2.28 | 2.72 |
| 01-04-79 | | | 2.63 | 2.29 | 2.20 | 2.26 | 2.70 |
| 01-05-79 | | | 2.63 | 2.28 | 2.28 | 2.26 | 2.70 |
| 01-07-79 | | | 2.62 | 2.27 | 2.16 | 2.25 | 2.69 |
| 01-11-79 | | | 2.62 | 2.27 | 2.15 | 2.24 | 2.68 |
| 01-15-79 | | | 2.62 | 2.26 | 2.15 | 2.24 | 2.69 |
| 01-23-79 | | | 2.63 | 2.25 | 2.14 | 2.23 | 2.70 |
| 02-10-79 | | | 2.62 | 2.24 | 2.10 | 2.21 | 2.67 |
| 02-17-79 | | | 2.63 | 2.24 | 2.10 | 2.22 | 2.69 |
| 02-26-79 | | | 2.63 | 2.24 | 2.10 | 2.21 | 2.69 |
| 03-10-79 | | | 2.69 | 2.28 | 2.15 | 2.26 | 2.75 |
| 03-16-79 | | | 2.68 | 2.29 | 2.15 | 2.27 | 2.74 |
| 03-24-79 | | | 2.68 | 2.27 | 2.14 | 2.26 | 2.73 |
| 03-31-79 | | | 2.69 | 2.46 | 2.14 | 2.33 | 2.74 |
| 04-07-79 | | | 2.67 | 2.26 | 2.12 | 2.24 | 2.72 |
| 04-27-79 | | | 2.67 | 2.42 | 2.11 | 2.31 | 2.72 |

TABLE XIV (Continued)

| Date | Station | Undisturbed Ground el, ft | E0+58 | E0+36 | E0+06 | W0+36 | W0+58 |
|---|---|---|---|---|---|---|---|
| | | | Settlement Readings, MSL, ft | | | | |
| | | | S-2-1 | S-2-2 | S-2-3 | S-2-4 | S-2-5 |
| 11-27-78 | 2+00 | 1.0 | 2.32 | 1.86 | 2.17 | 2.34 | 2.45 |
| 11-28-78 | | | 2.31 | 1.86 | 2.17 | 2.34 | 2.45 |
| 11-29-78 | | | 2.21 | 1.79 | 2.10 | 2.28 | 2.37 |
| 11-30-78 | | | 2.26 | 1.82 | 2.16 | 2.31 | 2.41 |
| 12-01-78 | | | 2.26 | 1.78 | 2.09 | 2.27 | 2.42 |
| 12-04-78 | | | 2.27 | 1.83 | 2.16 | 2.32 | 2.41 |
| 12-05-78 | | | 2.21 | 1.78 | 2.09 | 2.27 | 2.36 |
| 12-06-78 | | | 2.20 | 1.77 | 2.08 | 2.27 | 2.36 |
| 12-07-78 | | | 2.21 | 1.79 | 2.10 | 2.28 | 2.37 |
| 12-08-78 | | | 2.19 | 1.77 | 2.08 | 2.27 | 2.35 |
| 12-11-78 | | | 2.17 | 1.77 | 2.07 | 2.27 | 2.35 |
| 12-12-78 | | | 2.14 | 1.75 | 2.05 | 2.26 | 2.33 |
| 12-13-78 | | | 2.12 | 1.74 | 2.03 | 2.26 | 2.32 |
| 12-14-78 | | | 2.12 | 1.75 | 2.03 | 2.26 | 2.33 |
| 12-15-78 | | | 2.10 | 1.73 | 2.01 | 2.22 | 2.32 |
| 12-17-78 | | | 2.08 | 1.72 | 1.96 | 2.20 | 2.29 |
| 12-18-78 | | | 2.05 | 1.70 | 2.00 | 2.18 | 2.28 |
| 12-20-78 | | | 2.04 | 1.69 | 1.89 | 2.19 | 2.26 |
| 12-21-78 | | | 2.06 | 1.69 | 1.95 | 2.20 | 2.27 |
| 12-22-78 | | | 2.06 | 1.70 | 1.94 | 2.20 | 2.27 |
| 12-26-78 | | | 2.04 | 1.68 | 1.89 | 2.19 | 2.24 |
| 12-28-78 | | | 2.04 | 1.69 | 1.89 | 2.19 | 2.24 |
| 12-30-78 | | | 2.03 | 1.68 | 1.86 | 2.18 | 2.24 |
| 12-31-78 | | | 1.98 | 1.68 | 1.85 | 2.18 | 2.24 |
| 01-02-79 | | | 2.03 | 1.67 | 1.83 | 2.18 | 2.23 |
| 01-04-79 | | | 2.02 | 1.66 | 1.82 | 2.17 | 2.22 |
| 01-05-79 | | | 2.01 | 1.65 | 1.81 | 2.16 | 2.22 |
| 01-07-79 | | | 2.01 | 1.65 | 1.79 | 2.15 | 2.22 |
| 01-11-79 | | | 1.97 | 1.64 | 1.77 | 2.13 | 2.22 |
| 01-15-79 | | | 2.00 | 1.62 | 1.76 | 2.12 | 2.20 |
| 01-23-79 | | | 2.01 | 1.69 | 1.74 | 2.11 | 2.21 |
| 02-10-79 | | | 2.01 | 1.66 | 1.70 | 2.08 | 2.19 |
| 02-17-79 | | | 2.01 | 1.59 | 1.68 | 2.09 | 2.19 |
| 02-26-79 | | | 2.01 | 1.65 | 1.68 | 2.09 | 2.20 |
| 03-10-79 | | | 2.07 | 1.71 | 1.74 | 2.14 | 2.25 |
| 03-16-79 | | | 2.04 | 1.70 | 1.73 | 2.13 | 2.24 |
| 03-24-79 | | | 2.07 | 1.63 | 1.72 | 2.13 | 2.24 |
| 03-31-79 | | | 2.07 | 1.70 | 1.72 | 2.13 | 2.24 |
| 04-07-79 | | | 2.05 | 1.62 | 1.70 | 2.11 | 2.23 |
| 04-27-79 | | | 2.03 | 1.67 | 1.60 | 2.22 | 2.30 |

TABLE XIV (Continued)

| Date | Station | Undisturbed Ground el, ft | E0+58 | E0+36 | E0+06 | W0+36 | W0+58 |
|---|---|---|---|---|---|---|---|
| | | | Settlement Readings, MSL, ft | | | | |
| | | | S-3-1 | S-3-2 | S-3-3 | S-3-4 | S-3-5 |
| 12-01-78 | 3+00 | 0.9 | 2.20 | 2.34 | 2.19 | 1.93 | 2.26 |
| 12-05-78 | | | 2.21 | 2.36 | 2.21 | 1.95 | 2.28 |
| 12-06-78 | | | 2.20 | 2.35 | 2.20 | 1.95 | 2.28 |
| 12-07-78 | | | 2.22 | 2.36 | 2.20 | 1.93 | 2.28 |
| 12-08-78 | | | 2.19 | 2.33 | 2.17 | 1.92 | 2.27 |
| 12-11-78 | | | 2.18 | 2.31 | 2.14 | 1.90 | 2.26 |
| 12-12-78 | | | 2.17 | 2.29 | 2.11 | 1.88 | 2.24 |
| 12-13-78 | | | 2.16 | 2.28 | 2.09 | 1.87 | 2.27 |
| 12-14-78 | | | 2.17 | 2.28 | 2.09 | 1.88 | 2.24 |
| 12-15-78 | | | 2.15 | 2.26 | 2.06 | 1.86 | 2.22 |
| 12-17-78 | | | 2.13 | 2.24 | 2.01 | 1.84 | 2.20 |
| 12-18-78 | | | 2.12 | 2.22 | 1.99 | 1.83 | 2.19 |
| 12-20-78 | | | 2.11 | 2.21 | 1.94 | 1.73 | 2.15 |
| 12-21-78 | | | 2.13 | 2.22 | 2.02 | 1.76 | 2.15 |
| 12-22-78 | | | 2.11 | 2.21 | 1.99 | 1.73 | 2.15 |
| 12-26-78 | | | 2.09 | 2.19 | 1.92 | 1.71 | 2.15 |
| 12-28-78 | | | 2.05 | 2.20 | 1.88 | 1.67 | 2.13 |
| 12-30-78 | | | 2.08 | 2.22 | 1.89 | 1.70 | 2.13 |
| 12-31-78 | | | 2.07 | 2.21 | 1.87 | 1.69 | 2.13 |
| 01-02-79 | | | 2.07 | 2.20 | 1.86 | 1.68 | 2.13 |
| 01-04-79 | | | 2.07 | 2.20 | 1.85 | 1.68 | 2.13 |
| 01-05-79 | | | 2.07 | 2.21 | 1.84 | 1.66 | 2.13 |
| 01-07-79 | | | 2.06 | 2.17 | 1.83 | 1.65 | 2.13 |
| 01-11-79 | | | 2.06 | 2.16 | 1.80 | 1.64 | 2.13 |
| 01-15-79 | | | 2.04 | 2.13 | 1.78 | 1.61 | 2.11 |
| 01-23-79 | | | 2.05 | 2.13 | 1.76 | 1.76 | 2.11 |
| 02-10-79 | | | 2.03 | 2.09 | 1.71 | 1.72 | 2.10 |
| 02-17-79 | | | 2.03 | 2.09 | 1.70 | 1.56 | 2.10 |
| 02-26-79 | | | 2.02 | 2.08 | 1.68 | 1.71 | 2.11 |
| 03-10-79 | | | 2.08 | 2.14 | 1.74 | 1.76 | 2.17 |
| 03-16-79 | | | 2.07 | 2.12 | 1.73 | 1.75 | 2.17 |
| 03-24-79 | | | 2.08 | 2.13 | 1.72 | 1.60 | 2.16 |
| 03-31-79 | | | 2.08 | 2.18 | 1.72 | 1.74 | 2.17 |
| 04-07-79 | | | 2.07 | 2.12 | 1.71 | 1.57 | 2.16 |
| 04-27-79 | | | 2.06 | 2.14 | 1.68 | 1.69 | 2.14 |

TABLE XIV (Continued)

| Date | Station | Undisturbed Ground el, ft | E0+58 | E0+36 | E0+06 | W0+36 | W0+58 |
|---|---|---|---|---|---|---|---|
| | | | Settlement Readings, MSL, ft | | | | |
| | | | S-4-1 | S-4-2 | S-4-3 | S-4-4 | S-4-5 |
| 12-08-78 | 4+00 | 0.9 | 1.67 | 2.81 | 2.14 | 2.09 | 1.09 |
| 12-11-78 | | | 1.65 | 2.79 | 2.13 | 2.05 | 1.05 |
| 12-12-78 | | | 1.62 | 2.76 | 2.11 | 2.04 | 1.02 |
| 12-13-78 | | | 1.61 | 2.78 | 2.10 | 2.03 | 1.00 |
| 12-14-78 | | | 1.62 | 2.76 | 2.08 | 1.00 | 0.94 |
| 12-15-78 | | | 1.58 | 2.74 | 2.05 | 1.97 | 0.90 |
| 12-17-78 | | | 1.55 | 2.71 | 2.01 | 1.94 | 0.85 |
| 12-18-78 | | | 1.56 | 2.70 | 1.99 | 1.93 | 0.84 |
| 12-20-78 | | | 1.52 | 2.67 | 1.94 | 1.87 | 0.80 |
| 12-21-78 | | | 1.53 | 2.69 | 2.03 | 1.87 | 0.81 |
| 12-22-78 | | | 1.51 | 2.67 | 2.00 | 1.84 | 0.79 |
| 12-26-78 | | | 1.49 | 2.65 | 1.97 | 1.80 | 0.74 |
| 12-28-78 | | | 1.53 | 2.70 | 1.98 | 1.82 | 0.70 |
| 12-30-78 | | | 1.43 | 2.63 | 1.91 | 1.76 | 0.70 |
| 12-31-78 | | | 1.47 | 2.63 | 1.91 | 1.75 | 0.71 |
| 01-02-79 | | | 1.46 | 2.62 | 1.89 | 1.85 | 0.70 |
| 01-04-79 | | | 1.45 | 2.59 | 1.86 | 1.73 | 0.67 |
| 01-05-79 | | | 1.45 | 2.59 | 1.85 | 1.72 | 0.67 |
| 01-07-79 | | | 1.45 | 2.58 | 1.85 | 1.72 | 0.67 |
| 01-11-79 | | | 1.43 | 2.55 | 1.81 | 1.68 | 0.62 |
| 01-15-79 | | | 1.43 | 2.53 | 1.79 | 1.66 | 0.61 |
| 01-23-79 | | | 1.42 | 2.49 | 1.77 | 1.64 | 0.56 |
| 02-10-79 | | | 1.40 | 2.46 | 1.73 | 1.60 | 0.51 |
| 02-17-79 | | | 1.41 | 2.45 | 1.72 | 1.60 | 0.51 |
| 02-26-79 | | | 1.41 | 2.45 | 1.72 | 1.60 | 0.50 |
| 03-10-79 | | | 1.48 | 2.52 | 1.79 | 1.67 | 0.53 |
| 03-16-79 | | | 1.47 | 2.47 | 1.77 | 1.65 | 0.54 |
| 03-24-79 | | | 1.45 | 2.50 | 1.77 | 1.64 | 0.52 |
| 03-31-79 | | | 1.47 | 2.49 | 1.76 | 1.81 | 0.52 |
| 04-07-79 | | | 1.44 | 2.47 | 1.74 | 1.62 | 0.50 |
| 04-27-79 | | | 1.43 | 2.45 | 1.73 | 1.76 | 0.48 |

TABLE XIV (Continued)

| Date | Station | Undisturbed Ground el, ft | E0+58 | E0+36 | E0+06 | W0+36 | W0+58 |
|---|---|---|---|---|---|---|---|
| | | | Settlement Readings, MSL, ft | | | | |
| | | | S-5-1 | S-5-2 | S-5-3 | S-5-4 | S-5-5 |
| 12-15-78 | 5+00 | 1.00 | -0.05 | -1.25 | -0.26 | 1.79 | 1.18 |
| 12-17-78 | | | -0.18 | -1.48 | -0.42 | 1.71 | 0.97 |
| 12-18-78 | | | -0.22 | -1.49 | -0.47 | -- | 0.95 |
| 12-20-78 | | | -0.31 | -1.61 | -0.60 | 1.64 | 0.88 |
| 12-21-78 | | | -0.35 | -1.65 | -0.60 | 1.66 | 0.87 |
| 12-22-78 | | | -0.39 | -1.64 | -0.65 | 1.64 | 0.86 |
| 12-26-78 | | | -0.35 | -1.70 | -0.75 | 1.65 | 0.79 |
| 12-28-78 | | | -0.37 | -1.78 | -0.81 | 1.60 | 0.82 |
| 12-30-78 | | | -0.52 | -1.76 | -0.87 | 1.72 | 0.71 |
| 12-31-78 | | | -0.44 | -1.42 | -0.89 | 1.53 | 0.71 |
| 01-02-79 | | | -0.46 | -1.95 | -0.94 | 1.69 | 0.69 |
| 01-04-79 | | | -0.58 | -1.84 | -0.99 | 1.74 | 0.66 |
| 01-05-79 | | | -0.56 | -1.89 | -1.00 | 1.47 | 0.65 |
| 01-07-79 | | | -0.58 | -1.82 | -1.04 | 1.45 | 0.66 |
| 01-11-79 | | | -0.74 | -1.98 | -1.12 | 1.38 | 0.61 |
| 01-15-79 | | | -0.79 | -2.02 | -1.16 | 1.35 | 0.59 |
| 01-23-79 | | | -0.77 | -2.09 | -1.25 | 1.48 | 0.54 |
| 02-10-79 | | | -0.88 | -2.23 | -1.39 | 1.37 | 0.45 |
| 02-17-79 | | | -0.97 | -2.27 | -1.44 | 1.14 | 0.42 |
| 02-26-79 | | | -0.93 | -2.31 | -1.49 | 1.11 | 0.40 |
| 03-10-79 | | | -0.89 | -2.28 | -1.48 | 1.14 | 0.43 |
| 03-16-79 | | | -0.93 | -2.32 | - | 1.10 | 0.41 |
| 03-24-79 | | | -0.94 | -2.36 | - | 1.06 | 0.39 |
| 03-31-79 | | | -0.98 | -2.40 | - | 1.22 | 0.37 |
| 04-07-79 | | | -1.00 | -2.43 | - | 1.00 | 0.34 |
| 04-27-79 | | | -1.05 | -2.42 | -1.85 | 0.90 | 0.29 |

AD-A107 556   ARMY ENGINEER WATERWAYS EXPERIMENT STATION VICKSBURG--ETC   F/G 13/13
DESIGN, CONSTRUCTION, AND ANALYSIS OF FABRIC-REINFORCED EMBANKM--ETC(U)
OCT 81   J FOWLER
UNCLASSIFIED   WES/TR/EL-81-7   NL

3 OF 3

END
DATE
FILMED
1-82
DTIC

TABLE XIV (Concluded)

| Date | Station | Undisturbed Ground el, ft | E0+58 | E0+36 | E0+06 | W0+36 | W0+58 |
|---|---|---|---|---|---|---|---|
| | | | Settlement Readings, MSL, ft | | | | |
| | | | S-6-1 | S-6-2 | S-6-3 | S-6-4 | S-6-5 |
| 12-20-78 | 6+00 | 0.5 | 0.97 | 0.50 | 1.90 | 1.81 | 1.58 |
| 12-21-78 | | | 0.22 | 0.64 | 1.87 | 1.67 | 1.27 |
| 12-22-78 | | | 0.08 | 0.59 | 1.86 | 1.76 | 1.12 |
| 12-26-78 | | | 0.02 | 0.53 | 1.73 | 1.69 | 1.16 |
| 12-28-78 | | | 0.03 | 0.53 | 1.61 | 1.55 | 1.12 |
| 12-30-78 | | | -0.04 | 0.48 | 1.42 | 1.64 | 1.05 |
| 12-31-78 | | | -0.15 | 0.48 | 1.35 | 1.52 | 1.03 |
| 01-02-79 | | | -0.08 | 0.44 | 1.29 | 1.45 | 1.03 |
| 01-04-79 | | | -0.22 | 0.38 | 1.20 | 1.47 | 1.03 |
| 01-05-79 | | | -0.24 | 0.36 | 1.17 | 1.40 | 0.90 |
| 01-07-79 | | | -0.25 | 0.36 | 1.12 | 1.42 | 1.00 |
| 01-11-79 | | | -0.22 | 0.26 | 0.99 | 1.37 | 0.97 |
| 01-15-79 | | | -0.36 | 0.21 | 0.90 | 1.34 | 0.95 |
| 01-23-79 | | | -0.41 | 0.12 | 0.80 | 1.30 | 0.93 |
| 02-10-79 | | | -0.44 | -0.04 | 0.61 | 1.24 | 0.90 |
| 02-17-79 | | | -0.57 | -0.09 | 0.55 | 1.23 | 0.89 |
| 02-26-79 | | | -0.52 | -0.14 | 0.38 | 1.21 | 0.94 |
| 03-10-79 | | | -0.49 | -0.13 | 0.49 | 1.28 | 0.97 |
| 03-16-79 | | | -0.53 | -0.17 | 0.44 | 1.25 | 0.94 |
| 03-24-79 | | | -0.55 | -0.22 | 0.40 | 1.33 | 0.94 |
| 03-31-79 | | | -0.48 | -0.26 | 0.36 | 1.39 | 0.94 |
| 04-07-79 | | | -0.61 | -0.30 | 0.31 | 1.21 | 0.92 |
| 04-27-79 | | | -0.70 | -0.40 | 0.20 | 1.32 | 0.95 |
| | | | S-7-1 | S-7-2 | S-7-3 | S-7-4 | S-7-5 |
| 12-28-78 | 7+00 | 0.9 | 0.69 | 0.90 | 1.39 | 0.94 | -- |
| 12-30-78 | | | 0.62 | 0.80 | 1.24 | 0.91 | -- |
| 12-31-78 | | | 0.62 | 0.75 | 1.27 | 0.87 | -- |
| 01-02-79 | | | 0.60 | 0.72 | 1.14 | 0.85 | -- |
| 01-04-79 | | | 0.57 | 0.69 | 1.10 | 0.74 | 1.89 |
| 01-05-79 | | | 0.56 | 0.67 | 1.08 | 0.81 | 1.87 |
| 01-07-79 | | | 0.57 | 0.68 | 1.08 | 0.81 | 1.89 |
| 01-11-79 | | | 0.52 | 0.63 | 1.03 | 0.75 | 1.84 |
| 01-15-79 | | | 0.48 | 0.59 | 1.00 | 0.72 | 1.88 |
| 01-23-79 | | | 0.46 | 0.56 | 0.97 | 0.70 | 1.81 |
| 02-10-79 | | | 0.51 | 0.50 | 0.93 | 0.67 | 1.79 |
| 02-17-79 | | | 0.42 | 0.49 | 0.90 | 0.66 | 1.75 |
| 02-26-79 | | | 0.50 | 0.48 | 0.91 | 0.66 | 1.79 |
| 03-10-79 | | | 0.57 | 0.56 | 0.99 | 0.74 | 1.88 |
| 03-16-79 | | | 0.48 | 0.56 | 0.96 | 0.72 | 1.86 |
| 03-24-79 | | | 0.58 | 0.54 | 0.97 | 0.72 | 1.86 |
| 03-31-79 | | | 0.48 | 0.54 | 0.97 | 0.71 | 1.86 |
| 04-07-79 | | | 0.48 | 0.52 | 0.94 | 0.69 | 1.84 |
| 04-27-79 | | | 0.52 | 0.48 | 0.90 | 0.65 | 1.80 |

TABLE XV

PIEZOMETER READINGS, PINTO PASS TEST SECTION

| | | Piezometer Tip Elevations, MSL, ft | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| | | 0+00 | | | | W0+58 | | | |
| | | -30 | -20 | -10 | -5 | -30 | -20 | -10 | -5 |
| Date | Station | Pore Pressure, MSL, ft | | | | | | | |
| | | P-1-1 | P-1-2 | P-1-3 | P-1-4 | P-2-1 | P-2-2 | P-2-3 | P-2-4 |
| 12-14-78 | 1+00 | 1.72 | 1.70 | 1.87 | 3.24 | 1.50 | 1.40 | 2.0 | 1.50 |
| 12-17-78 | | 1.69 | 1.67 | 1.74 | 3.26 | 1.45 | 1.44 | 1.99 | 1.35 |
| 12-20-78 | | 2.00 | 1.98 | 1.78 | 3.88 | 1.66 | 1.56 | 2.20 | 1.58 |
| 12-22-78 | | 1.75 | 1.66 | 1.74 | 3.65 | 1.35 | 1.35 | 1.85 | 1.48 |
| 12-26-78 | | 1.62 | 1.44 | 1.44 | 3.71 | 1.40 | 0.66 | 1.37 | 1.26 |
| 12-28-78 | | 1.61 | 1.44 | 1.34 | 3.28 | 1.25 | 1.26 | 1.34 | 1.22 |
| 12-31-78 | | 1.75 | 1.44 | 1.64 | 3.64 | 1.35 | 1.36 | 2.20 | 1.48 |
| 01-03-79 | | 1.35 | 1.20 | 1.19 | 3.26 | 1.12 | 1.13 | 1.10 | 1.10 |
| 01-05-79 | | 1.37 | 1.28 | 1.22 | 3.72 | 1.27 | 1.21 | 1.64 | 1.21 |
| 01-07-79 | | 1.86 | 1.30 | 1.46 | 4.34 | 1.50 | 0.55 | 2.06 | 1.48 |
| 01-11-79 | | 1.50 | 0.99 | 1.42 | 3.84 | 1.34 | 1.10 | 2.04 | 1.44 |
| 01-15-79 | | 1.07 | 1.24 | 1.25 | 3.29 | 1.02 | 1.11 | 0.27 | 1.16 |
| 01-23-79 | | 1.27 | 1.26 | 1.26 | 4.00 | 1.20 | 1.25 | 1.86 | 1.28 |
| 02-10-79 | | 1.41 | 1.43 | 1.41 | 3.66 | 1.22 | 1.23 | 1.13 | 1.12 |
| 02-17-79 | | 1.40 | 1.67 | 1.14 | 1.06 | 0.94 | 2.11 | 1.43 | 0.88 |
| 02-26-79 | | 1.33 | 1.58 | 1.09 | 0.94 | 0.91 | 1.98 | 1.21 | 0.81 |
| 03-24-79 | | 1.85 | 1.85 | 1.81 | 3.98 | 1.57 | 1.57 | 1.56 | 1.54 |
| 03-31-79 | | 1.75 | 1.75 | 1.72 | 3.38 | 1.62 | 1.65 | 1.62 | 1.60 |
| 04-07-79 | | 1.53 | 1.58 | 1.56 | 3.37 | 1.41 | 1.41 | 1.65 | 0.87 |
| 04-27-79 | | 2.30 | 2.29 | 2.31 | 3.56 | 2.24 | 2.19 | 2.21 | 2.09 |
| | | W0+06 | | | | E0+58 | | | |
| | | P-3-1 | P-3-2 | P-3-3 | P-3-4 | P-4-1 | P-4-2 | P-4-3 | P-4-4 |
| 12-14-78 | 2+00 | 1.59 | 1.60 | 1.85 | 2.52 | 1.56 | 1.46 | 1.65 | 2.30 |
| 12-17-78 | | 1.56 | 1.51 | 1.81 | 2.85 | 1.43 | 1.45 | 1.56 | 2.22 |
| 12-20-78 | | 1.86 | 1.78 | 2.15 | 3.39 | 1.67 | 1.60 | 1.78 | 2.40 |
| 12-22-78 | | 1.52 | 1.63 | 1.88 | 3.27 | 1.48 | 1.60 | 1.61 | 2.22 |
| 12-26-78 | | 1.31 | 1.34 | 1.44 | 2.84 | 1.31 | 1.32 | 1.08 | 1.51 |
| 12-28-78 | | 1.04 | 1.12 | 1.61 | 3.00 | 1.29 | 1.28 | 1.27 | 1.21 |
| 12-31-78 | | 1.20 | 1.20 | 1.21 | 2.93 | 1.57 | 1.35 | 1.40 | 2.03 |
| 01-03-79 | | 0.87 | 0.86 | 1.25 | 2.51 | 1.15 | 1.15 | 1.13 | 1.87 |
| 01-05-79 | | 1.08 | 1.07 | 1.32 | 2.74 | 1.31 | 1.27 | 1.25 | 2.43 |
| 01-07-79 | | 1.49 | 1.50 | 1.79 | 3.10 | 1.61 | 1.46 | 1.62 | 2.66 |
| 01-11-79 | | 1.30 | 1.19 | 1.70 | 3.20 | 1.34 | 1.34 | 1.23 | 1.88 |
| 01-15-79 | | 0.49 | 0.70 | 0.36 | 2.69 | 1.24 | 1.24 | 1.21 | 1.21 |
| 01-23-79 | | 0.75 | 0.90 | 0.95 | 3.31 | 0.96 | 1.04 | 1.07 | 1.70 |
| 02-10-79 | | 0.91 | 0.94 | 1.32 | 3.32 | 0.62 | 1.02 | 0.97 | 1.71 |
| 02-17-79 | | 0.66 | 0.58 | 0.68 | 1.08 | 0.09 | 0.13 | 0.21 | 0.19 |
| 02-26-79 | | 1.21 | 0.73 | 0.63 | 1.00 | 0.00 | 0.03 | 0.30 | 0.15 |
| 03-24-79 | | 1.43 | 1.38 | 1.63 | 3.52 | 1.39 | 1.53 | 1.51 | 2.01 |
| 03-31-79 | | 1.49 | 1.51 | 1.69 | 3.08 | * | | | |
| 04-07-79 | | 0.99 | 1.05 | 0.99 | 3.06 | 1.25 | 1.26 | 1.25 | 1.61 |
| 04-27-79 | | 2.12 | 2.13 | 2.29 | 3.32 | 2.13 | 2.14 | 2.09 | 2.17 |

* Instrument failed

TABLE XV (Continued)

| | | Piezometer Tip Elevations, MSL, ft | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| | | W0+06 | | | | W0+58 | | | |
| | | -30 | -20 | -10 | -5 | -30 | -20 | -10 | -5 |
| | | Pore Pressure, MSL, ft | | | | | | | |
| Date | Station | P-5-1 | P-5-2 | P-5-3 | P-5-4 | P-6-1 | P-6-2 | P-6-3 | P-6-4 |
| 12-17-78 | 3+00 | 1.61 | 1.85 | 1.97 | 2.23 | 1.06 | 0.99 | 0.94 | 1.50 |
| 12-20-78 | | 2.11 | 2.44 | 2.44 | 2.91 | Broken | 0.57 | 0.88 | 1.67 |
| 12-22-78 | | 1.83 | 2.15 | 2.14 | 2.45 | 1.12 | 1.05 | 0.91 | 1.56 |
| 12-26-78 | | 1.35 | 1.61 | 1.68 | 1.96 | 0.85 | 0.86 | 0.86 | 0.84 |
| 12-28-78 | | 1.24 | 1.25 | 1.27 | 1.26 | 0.64 | 0.64 | 0.66 | 0.65 |
| 12-31-78 | | 1.33 | 1.18 | 1.18 | 1.04 | 1.03 | 0.90 | 0.85 | 1.40 |
| 01-03-79 | | 0.86 | 0.86 | 0.86 | 1.25 | 0.44 | 0.43 | 0.43 | 1.10 |
| 01-05-79 | | 1.17 | 1.48 | 0.97 | 2.11 | 0.41 | 0.87 | 0.62 | 1.37 |
| 01-07-79 | | 1.51 | 1.58 | 1.60 | 1.90 | 1.02 | 1.02 | 0.95 | 0.94 |
| 01-11-79 | | - | 1.30 | 1.34 | 1.41 | 0.85 | 0.79 | 0.62 | 1.41 |
| 01-15-79 | | 0.54 | 0.58 | 0.60 | 0.57 | 0.67 | 0.67 | 0.68 | 0.62 |
| 01-23-79 | | 0.80 | 0.88 | 0.94 | 0.94 | 0.59 | 0.56 | 0.56 | 1.37 |
| 02-10-79 | | 0.01 | 0.17 | -0.03 | 0.24 | 0.70 | 0.63 | 0.57 | 0.61 |
| 02-17-79 | | 0.43 | 0.48 | 1.12 | 0.73 | -0.09 | -0.07 | 0.01 | -0.01 |
| 02-26-79 | | 0.43 | 0.33 | 1.04 | 0.76 | -0.15 | -0.11 | -0.01 | -0.03 |
| 03-24-79 | | 1.25 | 1.09 | 1.28 | 1.43 | 1.00 | 0.96 | 0.90 | 1.53 |
| 03-31-79 | | 1.38 | 1.22 | 1.34 | 1.29 | 1.50 | 1.51 | 1.43 | 1.48 |
| 04-07-79 | | 0.97 | 0.82 | 0.96 | 0.95 | 0.85 | 0.29 | 0.76 | 1.07 |
| 04-27-79 | | 2.01 | 2.02 | 2.01 | 1.97 | 2.13 | 2.08 | 2.03 | 1.96 |
| | | W0+06 | | | | W0+58 | | | |
| | | P-7-1 | P-7-2 | P-7-3 | P-7-4 | P-8-1 | P-8-2 | P-8-3 | P-8-4 |
| 12-16-78 | 4+00 | 2.41 | 2.74 | 3.06 | 3.61 | 1.37 | 1.59 | 1.62 | 1.80 |
| 12-20-78 | | 3.90 | 4.49 | 4.76 | 4.91 | 1.73 | 1.94 | 1.91 | 2.00 |
| 12-22-78 | | 2.79 | 2.85 | 3.30 | 3.88 | 1.40 | 1.34 | 1.37 | 1.80 |
| 12-26-78 | | 1.74 | 1.69 | 2.29 | 3.60 | 1.10 | 1.09 | 1.32 | 1.43 |
| 12-28-78 | | 1.29 | 2.08 | 2.07 | 2.94 | 0.69 | 0.89 | 0.94 | 1.19 |
| 12-31-78 | | 2.06 | 2.20 | 2.35 | 3.54 | 1.33 | 1.31 | 1.15 | 1.15 |
| 01-03-79 | | 1.42 | 1.68 | 1.80 | 3.04 | 0.84 | 1.07 | 1.11 | 1.29 |
| 01-05-79 | | 1.33 | 2.00 | 2.02 | 3.11 | 1.22 | 1.19 | 1.17 | 1.28 |
| 01-07-79 | | 1.61 | 1.98 | 2.14 | 3.21 | 1.39 | 1.32 | 1.31 | 1.31 |
| 01-11-79 | | 1.55 | 1.74 | 4.85 | 3.03 | 0.86 | 1.05 | 0.94 | 1.29 |
| 01-15-79 | | 0.90 | 1.31 | 1.37 | 2.81 | 0.83 | 0.79 | 0.79 | 0.90 |
| 01-23-79 | | 0.94 | 0.74 | 0.75 | 3.17 | 0.00 | 0.44 | 0.45 | 1.48 |
| 02-10-79 | | 0.56 | 1.02 | 1.08 | 2.10 | 0.84 | 0.82 | 0.82 | 0.29 |
| 02-17-79 | | 0.63 | 0.49 | 0.69 | 0.93 | 0.70 | 0.74 | 0.67 | 0.72 |
| 02-26-79 | | 0.55 | 0.48 | 0.64 | 0.80 | 0.67 | 0.73 | 0.67 | 0.65 |
| 03-24-79 | | 1.47 | 1.47 | 1.63 | 3.26 | 1.38 | 1.30 | 1.31 | 1.33 |
| 03-31-79 | | 1.39 | 1.43 | 1.42 | 2.65 | | | | |
| 04-07-79 | | 0.96 | 0.86 | 0.89 | 2.69 | 1.09 | 1.12 | 1.07 | 1.29 |
| 04-27-79 | | 2.18 | 2.09 | 2.11 | 3.24 | 2.18 | 2.16 | 2.10 | 2.03 |

TABLE XV (Continued)

| | | Piezometer Tip Elevations, MSL, ft | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| | | W0+06 | | | | W0+58 | | | |
| | | -30 | -20 | -10 | -5 | -30 | -20 | -10 | -5 |
| | | Pore Pressure, MSL, ft | | | | | | | |
| Date | Station | P-9-1 | P-9-2 | P-9-3 | P-9-4 | P-10-1 | P-10-2 | P-10-3 | P-10-4 |
| 12-17-78 | 5+00 | 0.92 | 2.74 | 2.78 | 6.53 | 2.45 | 2.36 | 2.24 | 1.78 |
| 12-20-78 | | 2.47 | 4.29 | 4.44 | 9.00 | Broken | 2.80 | 2.71 | 2.28 |
| 12-22-78 | | 2.20 | 3.55 | 3.54 | 9.00 | 2.47 | 2.48 | 2.33 | 2.39 |
| 12-26-78 | | 1.55 | 2.56 | 2.55 | Broken | 1.01 | 2.72 | 2.72 | 1.84 |
| 12-28-78 | | 1.33 | 2.35 | 2.34 | 9.00 | 1.80 | 1.79 | 1.70 | 2.43 |
| 12-31-78 | | 1.78 | 2.62 | 2.63 | 9.00 | 1.89 | 1.55 | 1.62 | 2.16 |
| 01-03-79 | | 0.78 | 2.06 | 2.08 | 9.00 | 1.42 | 1.41 | 1.24 | 2.30 |
| 01-05-79 | | 0.84 | 2.13 | 1.87 | 10.64 | 1.63 | 1.57 | 2.02 | 2.37 |
| 01-07-79 | | 0.93 | 2.25 | 1.65 | 11.21 | 1.34 | 1.59 | 1.22 | 2.40 |
| 01-11-79 | | 1.20 | 1.30 | 1.32 | 9.80 | 1.33 | 1.62 | 1.48 | 2.60 |
| 01-15-79 | | 0.67 | 1.30 | 1.26 | 9.66 | 1.04 | 1.05 | 1.08 | 2.95 |
| 01-23-79 | | 0.35 | 0.83 | 0.11 | 5.37 | 1.21 | 1.18 | 1.07 | 2.56 |
| 02-10-79 | | 0.67 | 1.17 | 0.74 | 8.87 | 0.92 | 0.89 | 0.85 | 2.47 |
| 02-17-79 | | -0.02 | 0.52 | 0.55 | 8.31 | 0.91 | 0.43 | 0.70 | 0.70 |
| 02-26-79 | | -0.01 | 0.36 | 0.47 | 9.3 | 0.82 | 0.36 | 0.62 | 0.67 |
| 03-24-79 | | 1.42 | 1.48 | 1.43 | 7.29 | 1.21 | 1.19 | 1.21 | 2.27 |
| 03-31-79 | | 1.68 | 1.55 | 1.54 | | | | | |
| 04-07-79 | | 0.63 | 0.72 | 0.91 | 6.9 | 0.91 | 0.93 | 0.92 | 2.11 |
| 04-27-79 | | 2.27 | 2.24 | 2.15 | 6.65 | 2.16 | 2.12 | 2.09 | 2.02 |
| | | W0+06 | | | | W0+58 | | | |
| | | P-11-1 | P-11-2 | P-11-3 | P-11-4 | P-12-1 | P-12-2 | P-12-3 | P-12-4 |
| 12-21-78 | 6+00 | | | | | 4.70 | 4.85 | 5.00 | 5.00 |
| 12-22-78 | | | | | | 4.31 | 4.36 | 4.54 | 5.00 |
| 12-26-78 | | | | | | 3.08 | 3.07 | 3.34 | 5.00 |
| 12-28-78 | | | | | | 3.14 | 3.13 | 3.26 | 5.00 |
| 12-31-78 | | 3.72 | 3.63 | 3.35 | 9.00 | 3.27 | 3.28 | 3.31 | 5.00 |
| 01-03-79 | | 2.81 | 2.63 | 2.34 | 8.40 | 2.66 | 2.69 | 2.81 | 5.00 |
| 01-05-79 | | 2.86 | 2.58 | 2.29 | 9.62 | 2.89 | 2.81 | 3.11 | 10.53 |
| 01-07-79 | | 2.70 | 2.46 | 2.21 | 10.40 | 2.51 | 2.51 | 2.69 | 9.98 |
| 01-11-79 | | 2.27 | 2.08 | 2.01 | 7.65 | 2.32 | 2.40 | 2.40 | 1.37 |
| 01-15-79 | | 1.80 | 1.70 | 1.63 | 5.88 | 1.81 | 1.79 | 1.82 | 5.72 |
| 01-23-79 | | 1.79 | 1.91 | 1.12 | 5.89 | 1.80 | 1.80 | 1.81 | 1.64 |
| 02-10-79 | | 0.91 | 1.43 | 1.21 | 4.64 | 0.79 | 1.22 | 1.45 | 5.35 |
| 02-17-79 | | 0.65 | 1.01 | 0.27 | 0.69 | 0.86 | 1.20 | 0.88 | 5.20 |
| 02-26-79 | | 0.59 | 1.05 | 0.25 | 0.63 | 0.75 | 1.06 | 0.74 | 5.07 |
| 03-24-79 | | 1.55 | 1.52 | 1.46 | 4.87 | 2.33 | 2.65 | 1.69 | 1.45 |
| 03-31-79 | | | | | | | | | |
| 04-07-79 | | 0.48 | 1.05 | 1.23 | 1.63 | 0.40 | 0.39 | 0.40 | 1.24 |
| 04-27-79 | | 2.25 | 2.20 | 2.11 | 3.95 | 2.27 | 2.21 | 2.22 | 1.30 |

TABLE XV (Concluded)

| | | Piezometer Tip Elevations, MSL, ft | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| | | W0+06 | | | | W0+58 | | | |
| | | -30 | -20 | -10 | -5 | -30 | -20 | -10 | -5 |
| | | Pore Pressure, MSL, ft | | | | | | | |
| Date | Station | P-13-1 | P-13-2 | P-13-3 | P-13-4 | P-14-1 | P-14-2 | P-14-3 | P-14-4 |
| 12-21-78 | 7+00 | 2.74 | 2.65 | 2.73 | 2.95 | 1.98 | 1.39 | 1.60 | 2.02 |
| 01-03-79 | | 2.16 | 2.09 | 2.09 | 2.02 | 1.68 | 1.65 | 1.57 | 1.74 |
| 01-05-79 | | 2.13 | 2.11 | 2.14 | 2.21 | 1.56 | 1.57 | 1.49 | 1.62 |
| 01-07-79 | | 2.10 | 2.05 | 2.13 | 2.42 | 1.70 | 1.65 | 1.55 | 1.55 |
| 01-11-79 | | 1.65 | 1.84 | 1.86 | 2.10 | 1.57 | 1.56 | 1.58 | 1.82 |
| 01-15-79 | | 1.31 | 1.59 | 1.59 | 1.62 | 1.41 | 1.41 | 1.41 | 1.36 |
| 01-23-79 | | 1.59 | 1.45 | 1.43 | 1.44 | 1.30 | 1.30 | 1.39 | 1.78 |
| 02-10-79 | | 1.34 | 1.16 | 1.16 | 1.43 | 1.24 | 1.30 | 1.33 | 1.29 |
| 02-17-79 | | 1.37 | 1.10 | 1.08 | 1.53 | 0.78 | 0.95 | 0.86 | 0.76 |
| 02-26-79 | | 1.26 | 1.03 | 1.03 | 1.39 | 0.79 | 0.87 | 0.75 | 0.65 |
| 03-24-79 | | 1.55 | 1.50 | 1.57 | 1.75 | 1.52 | 1.54 | 1.61 | 1.51 |
| 03-31-79 | | | | | | | | | |
| 04-07-79 | | 1.39 | 1.30 | 1.39 | 1.37 | 1.37 | 1.36 | 1.35 | 1.35 |
| 04-27-79 | | 2.13 | 2.07 | 2.18 | 2.29 | 2.06 | 2.08 | 2.08 | 2.04 |

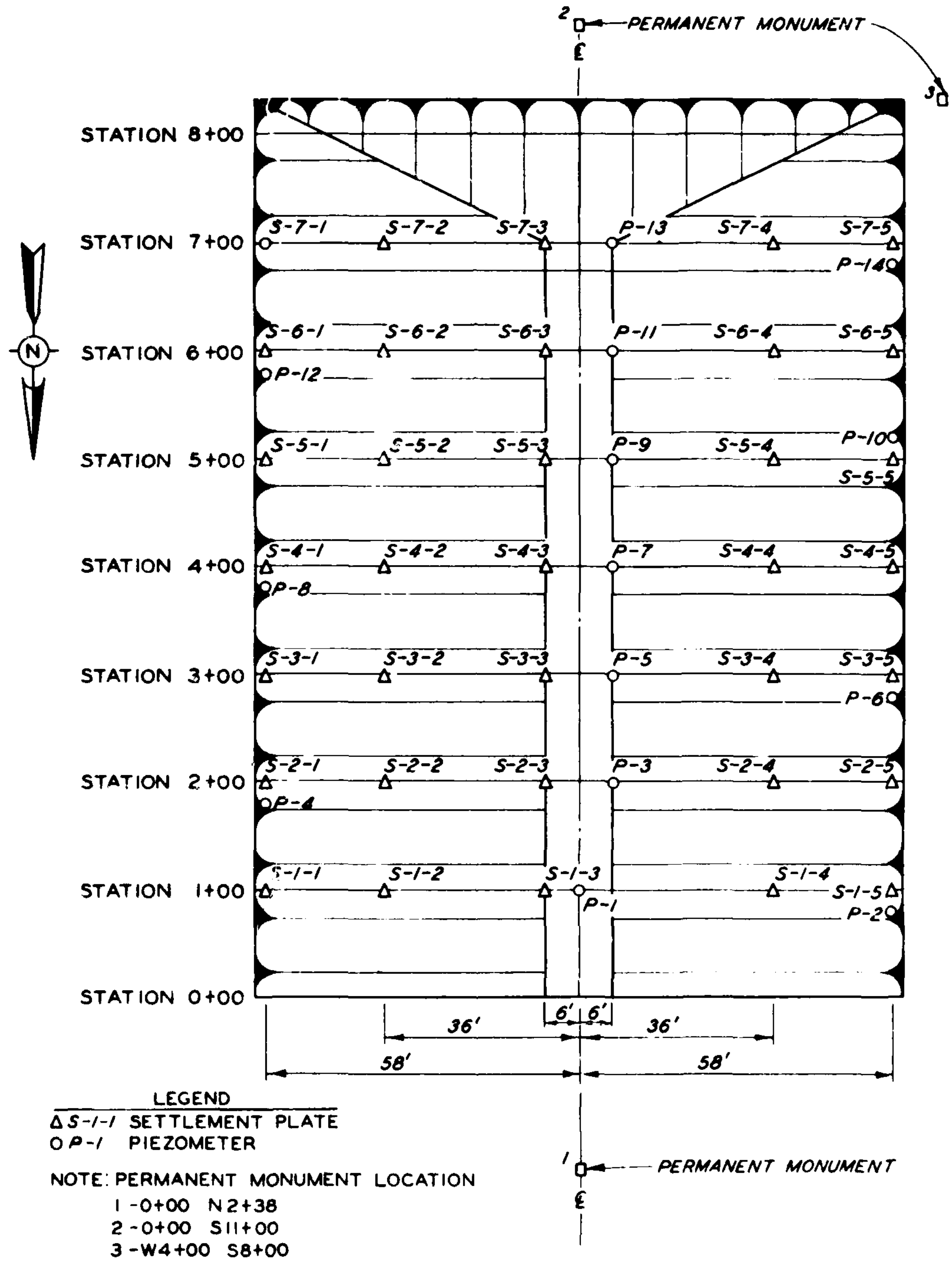



Figure 67. Locations of Settlement Plates, Piezometers, and Permanent Monuments

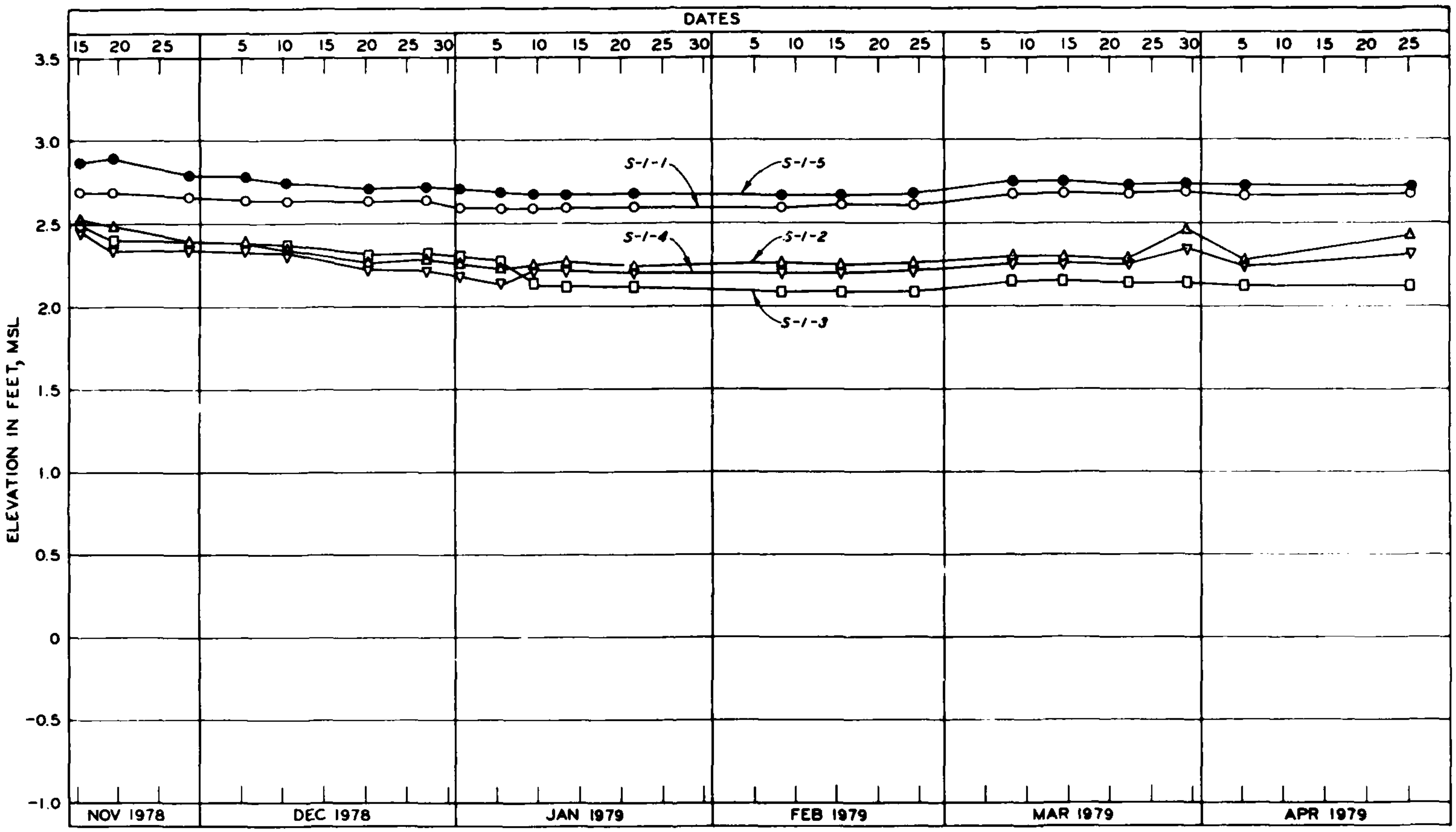



Figure 68. Settlement versus Time for Settlement Plates S-1-1 through S-1-5

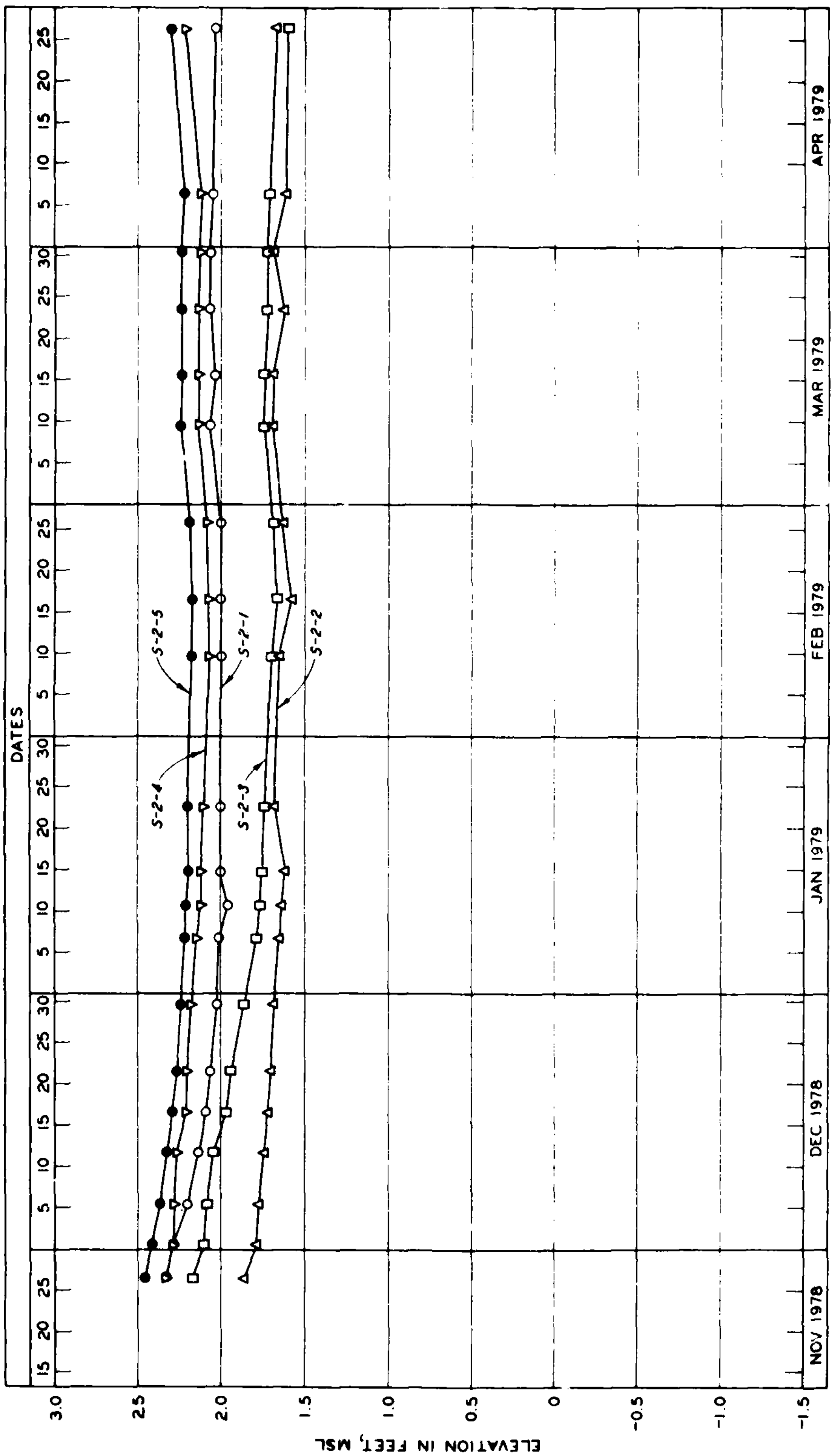



Figure 69. Settlement versus Time for Settlement Plates S-2-1 through S-2-5

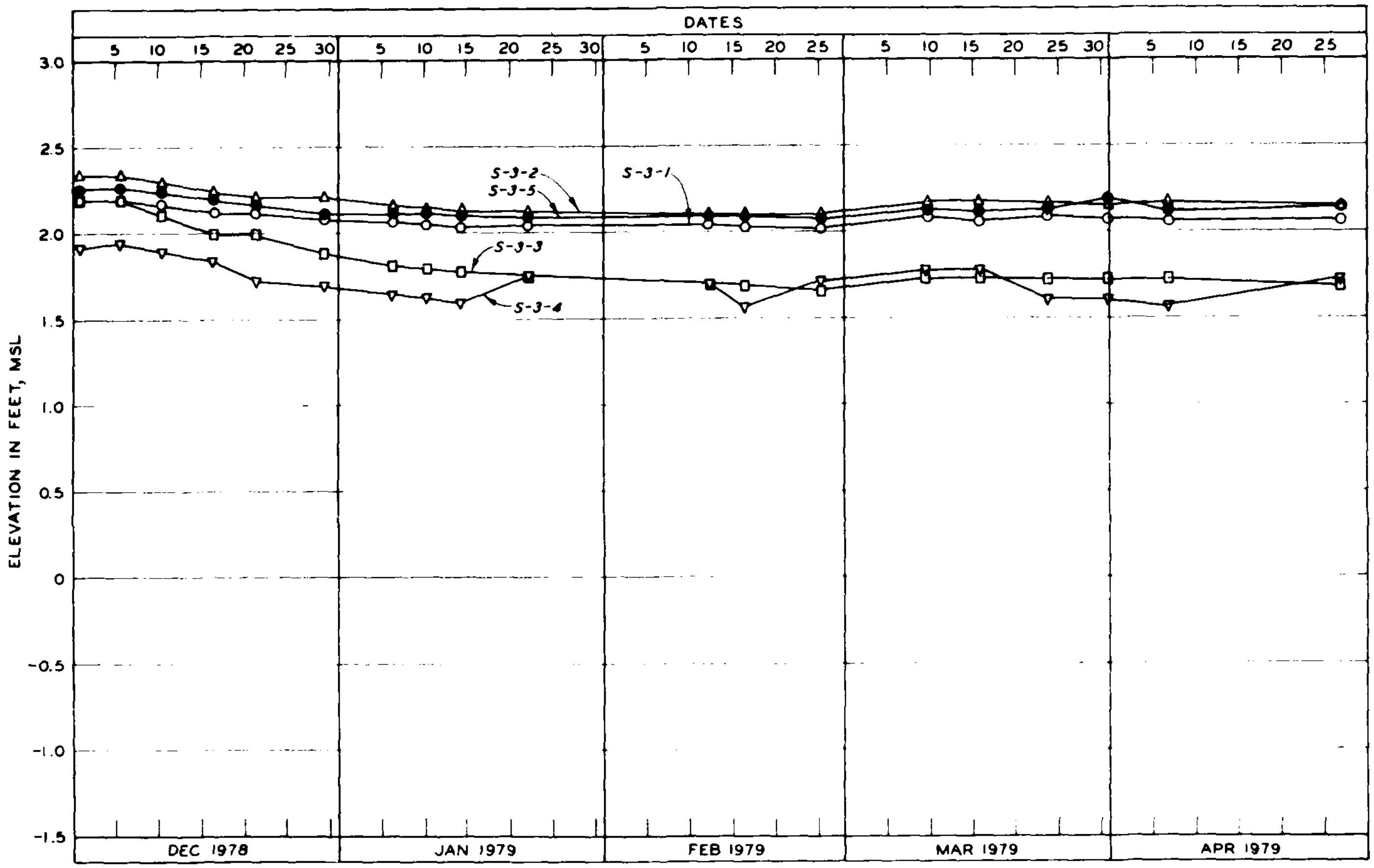



Figure 70. Settlement versus Time for Settlement Plates S-3-1 through S-3-5

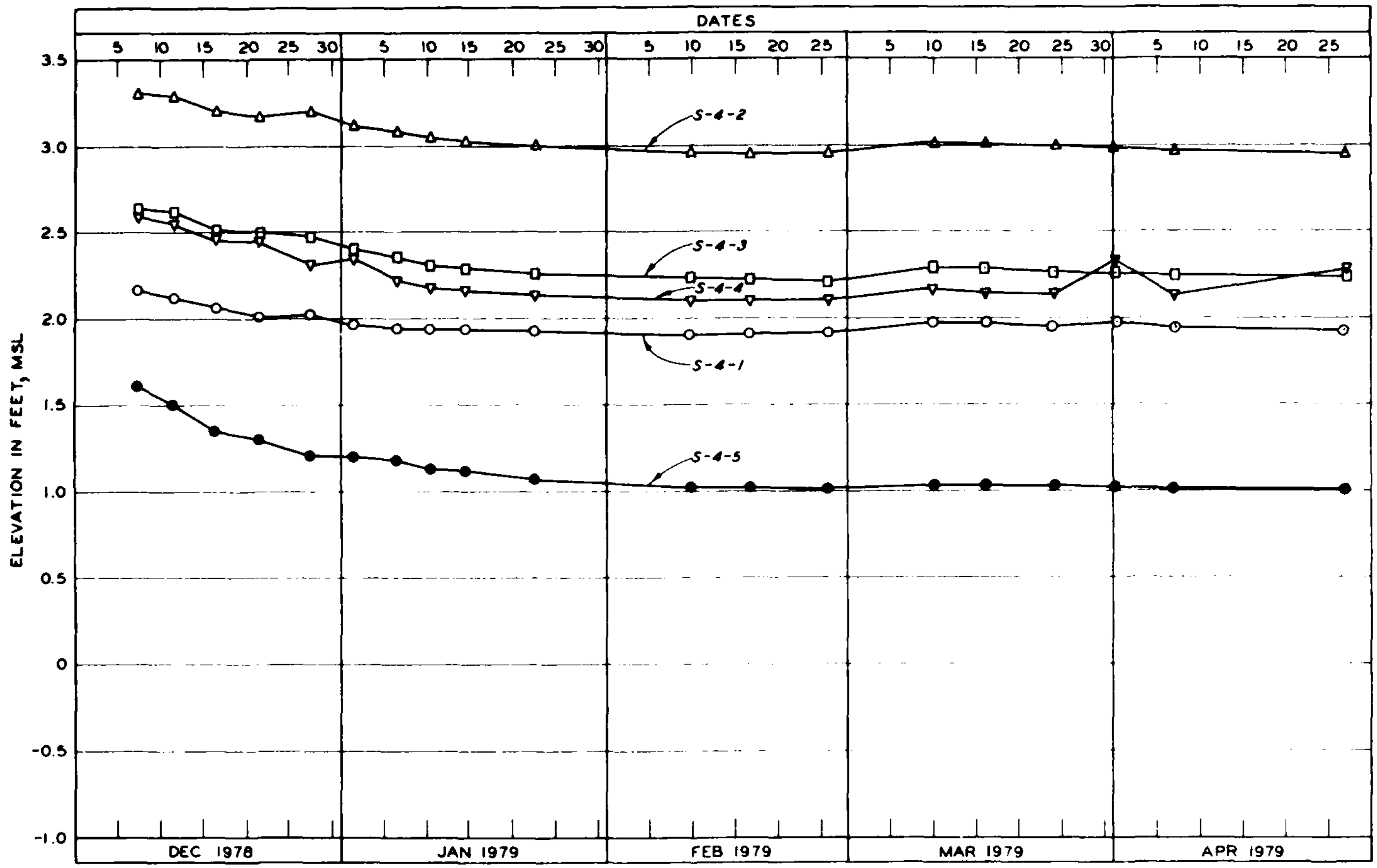



Figure 71. Settlement versus Time for Settlement Plates S-4-1 through S-4-5

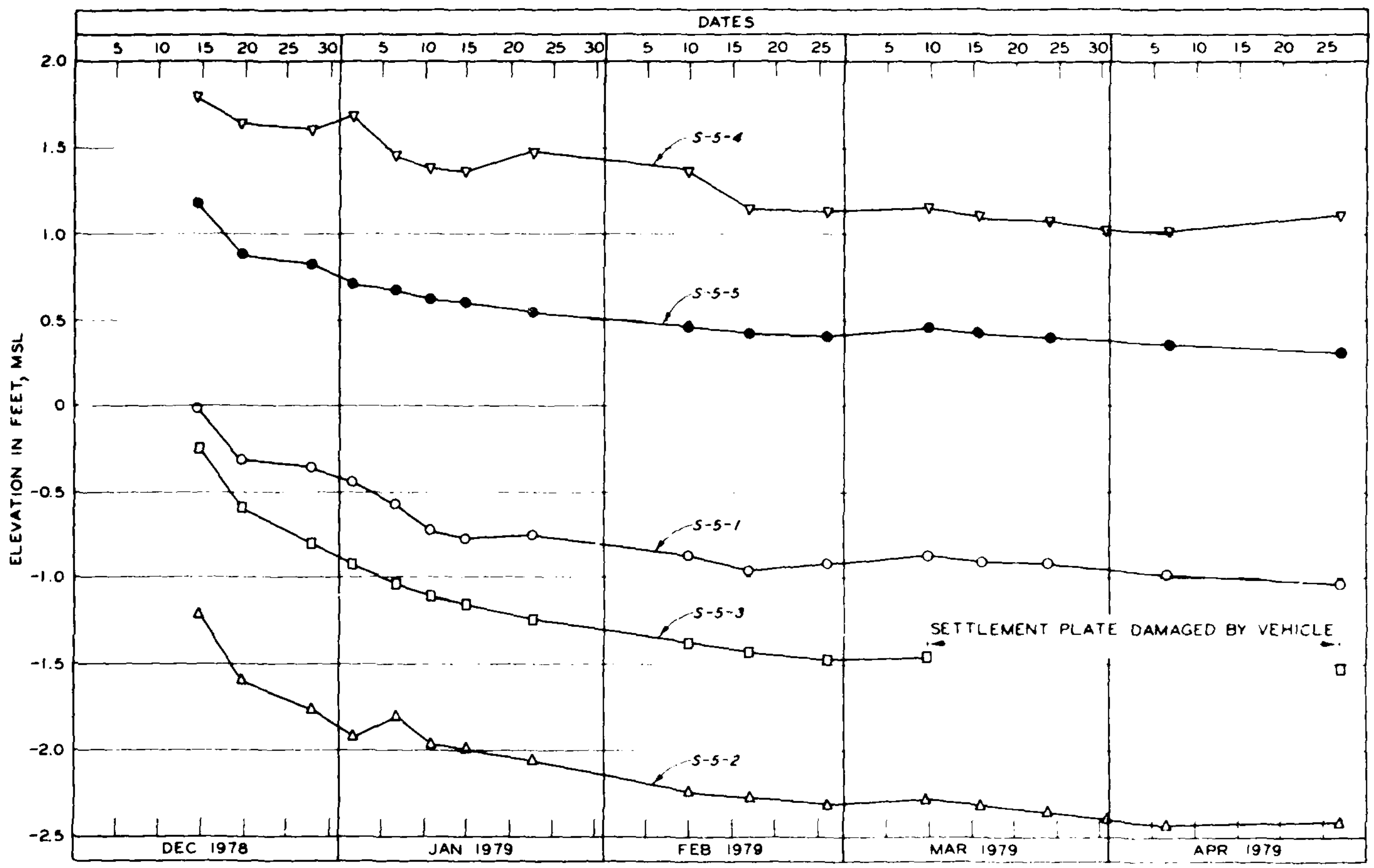



Figure 72. Settlement versus Time for Settlement Plates S-5-1 through S-5-5

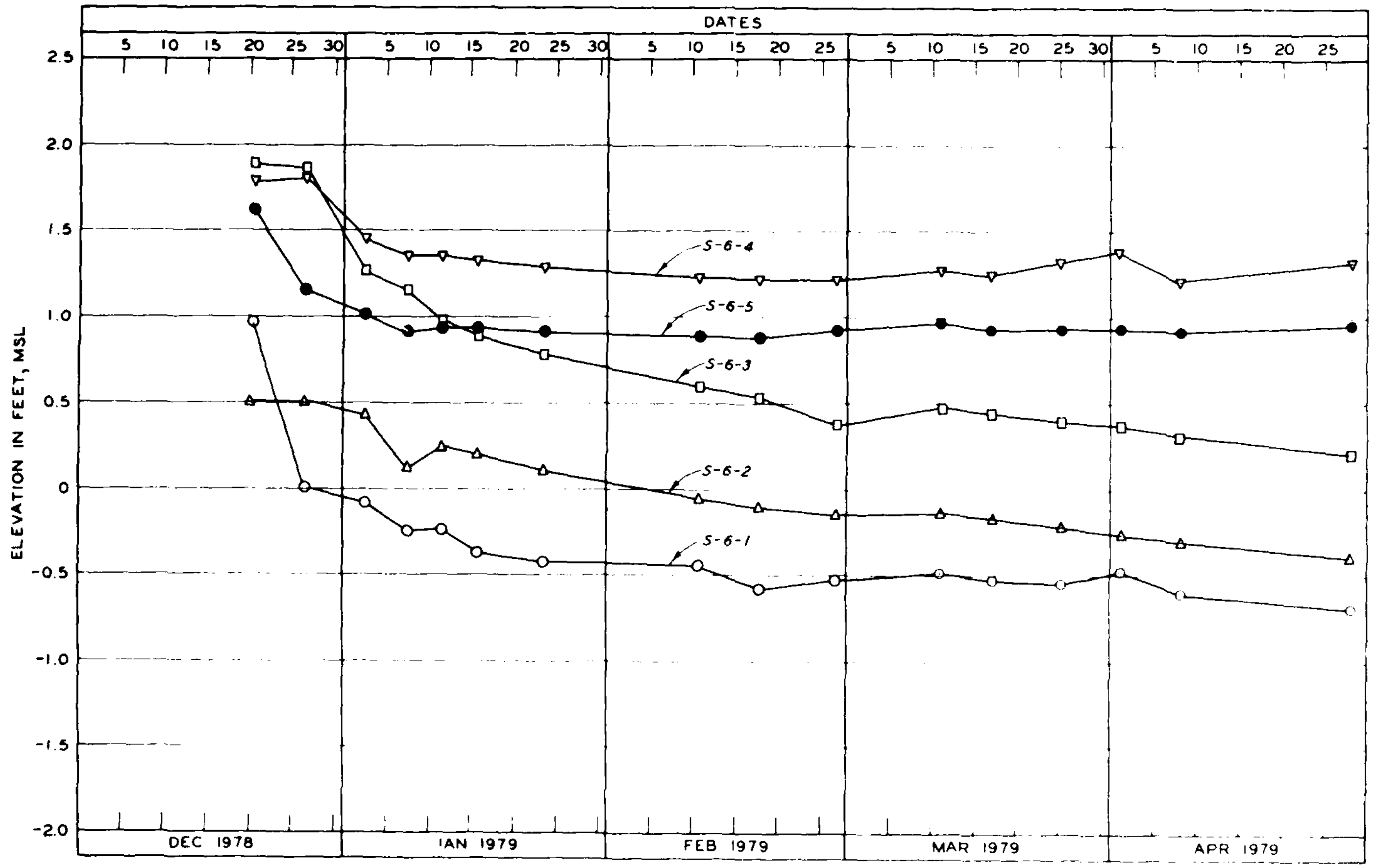



Figure 73. Settlement versus Time for Settlement Plates S-6-1 through S-6-5

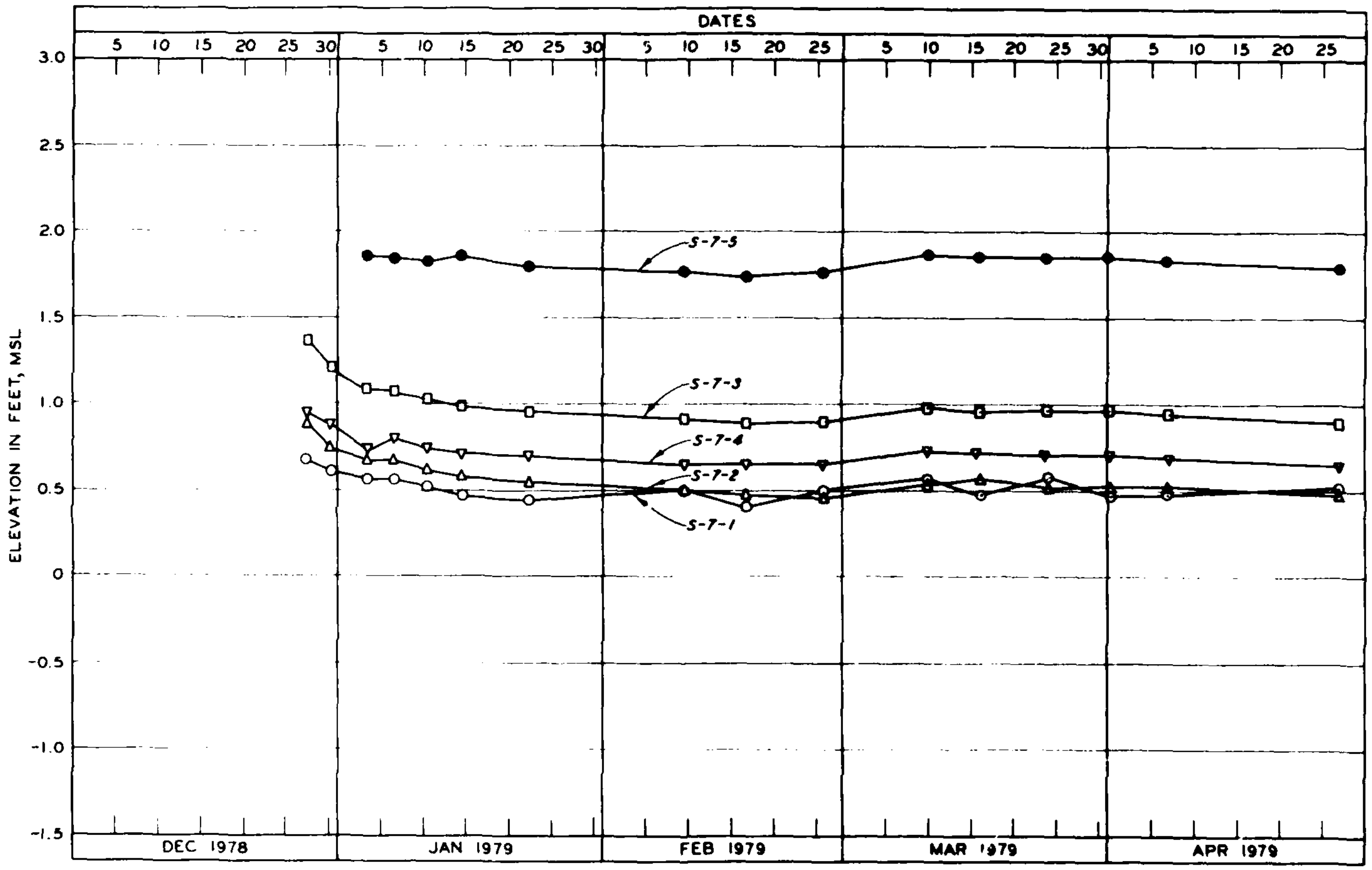



Figure 74. Settlement versus Time for Settlement Plates S-7-1 through S-7-5

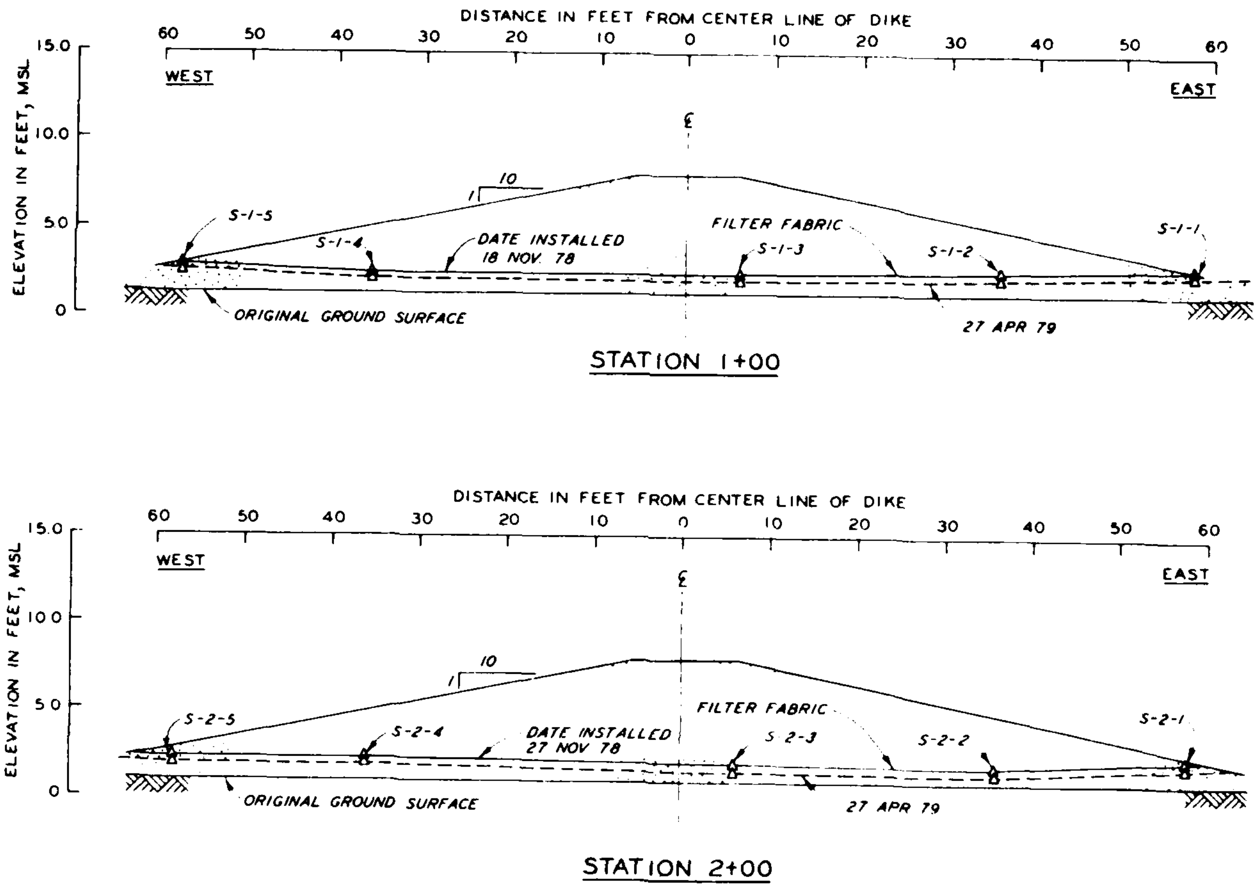



Figure 75. Settlement Profiles for Stations 1+00 and 2+00

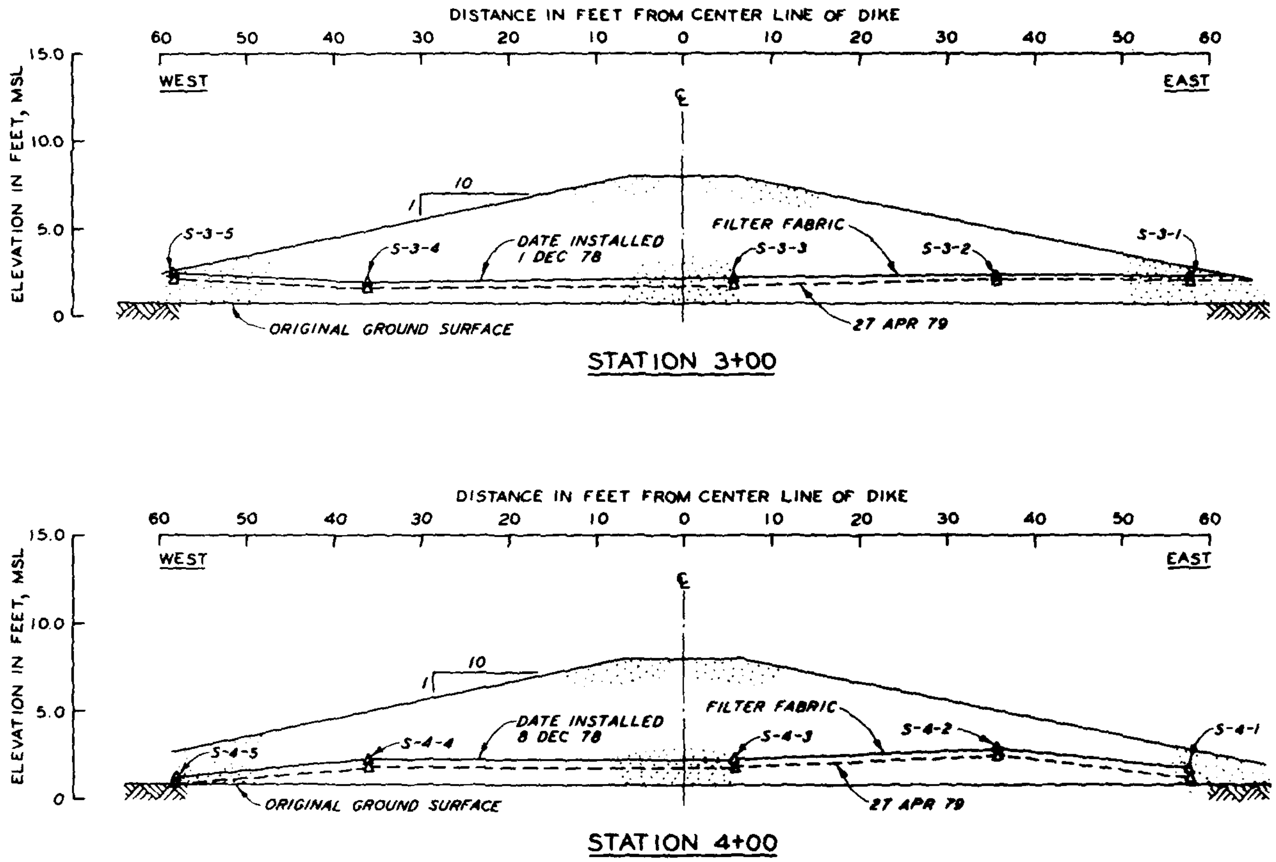



Figure 76. Settlement Profiles for Stations 3+00 and 4+00

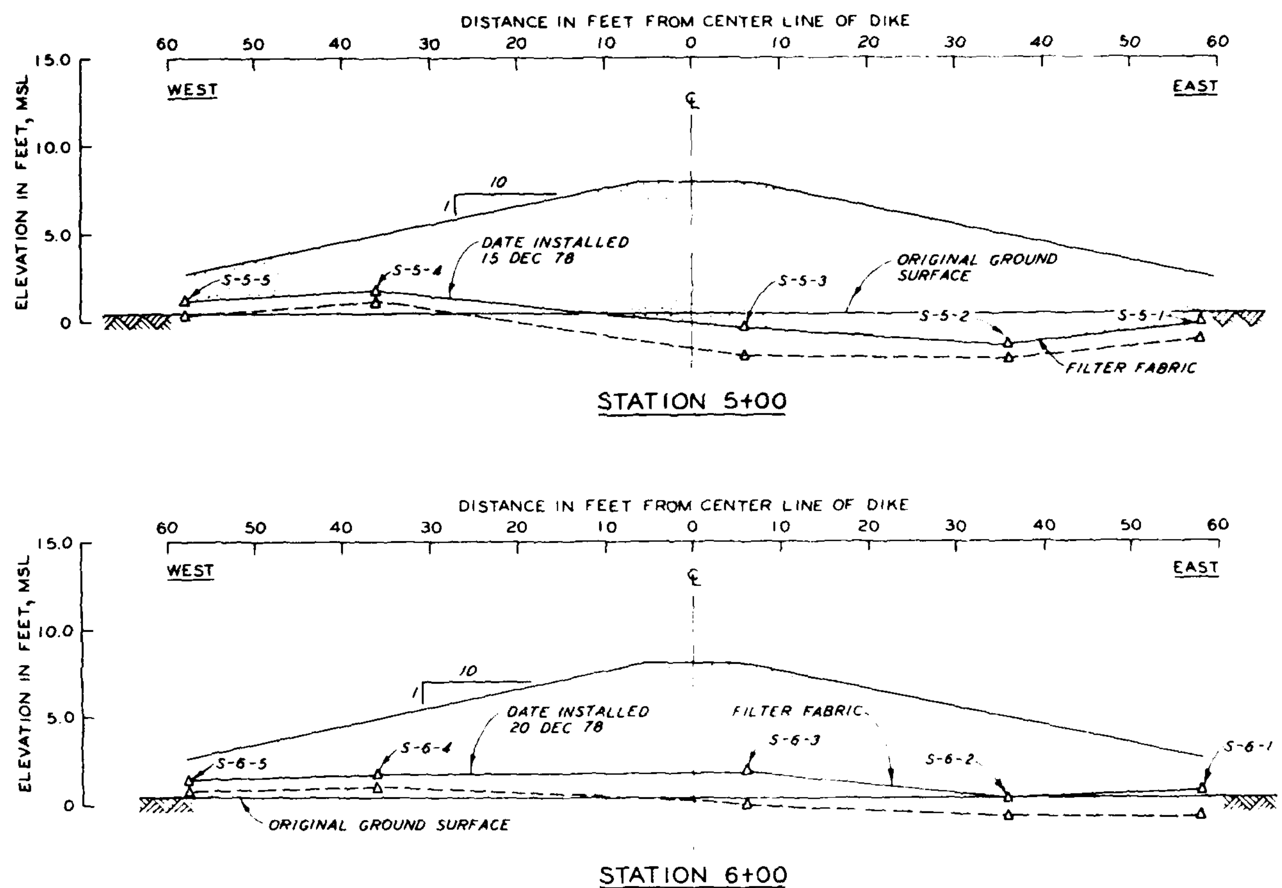



Figure 77. Settlement Profiles for Stations 5+00 and 6+00

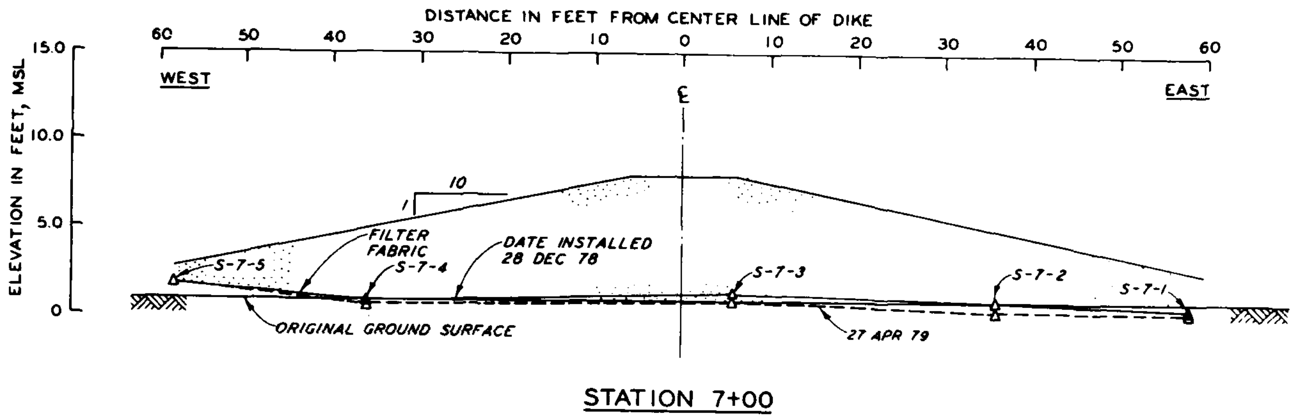



Figure 78. Settlement Profile for Station 7+00

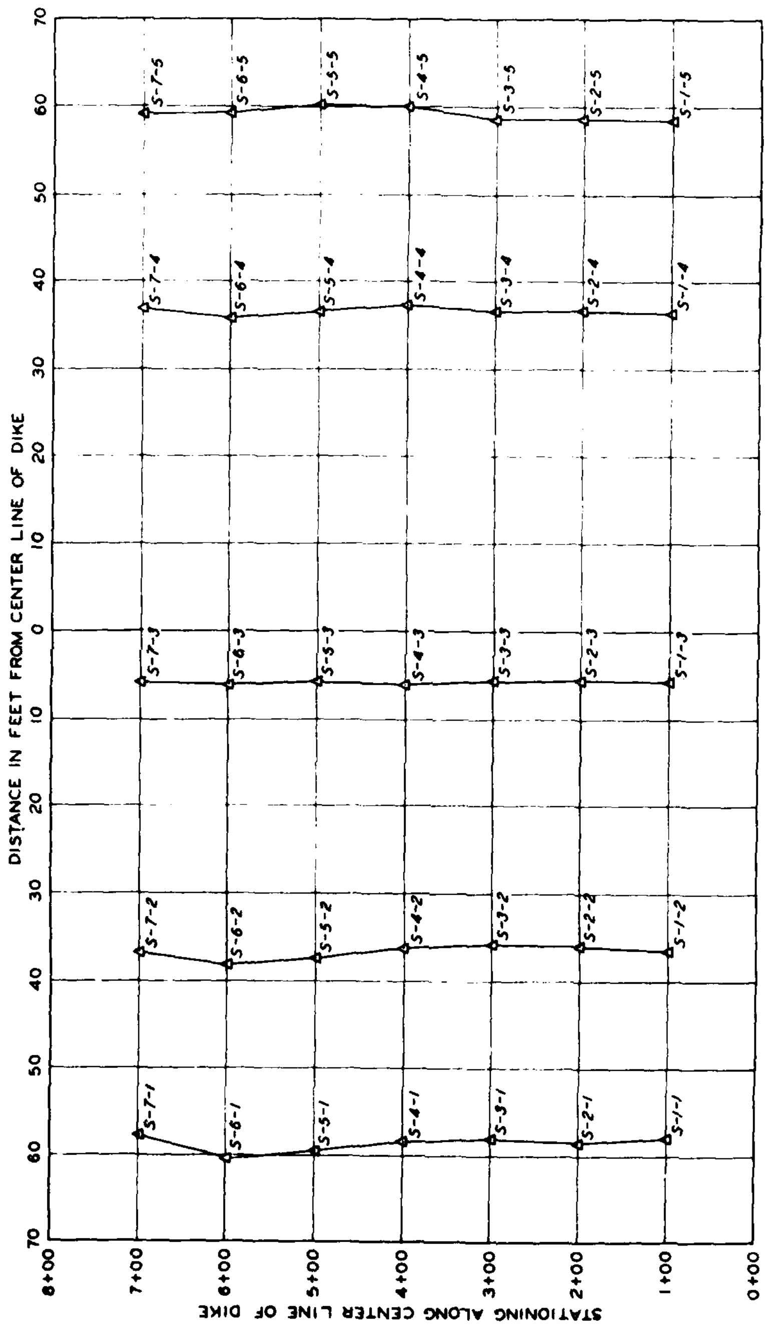



Figure 79. Plan View of Maximum Horizontal Displacement

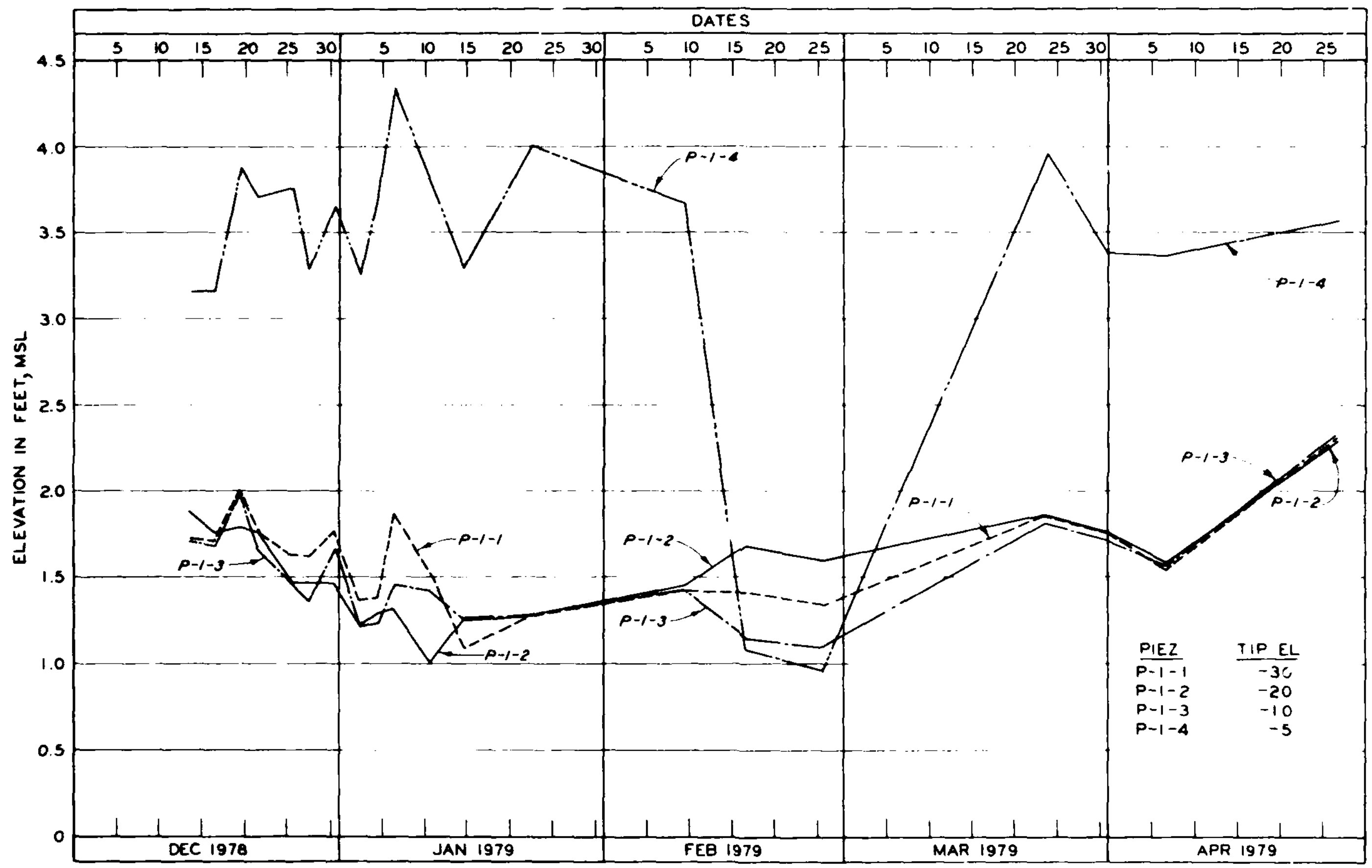



Figure 80. Pore Pressure versus Time for Piezometers P-1-1 through P-1-4

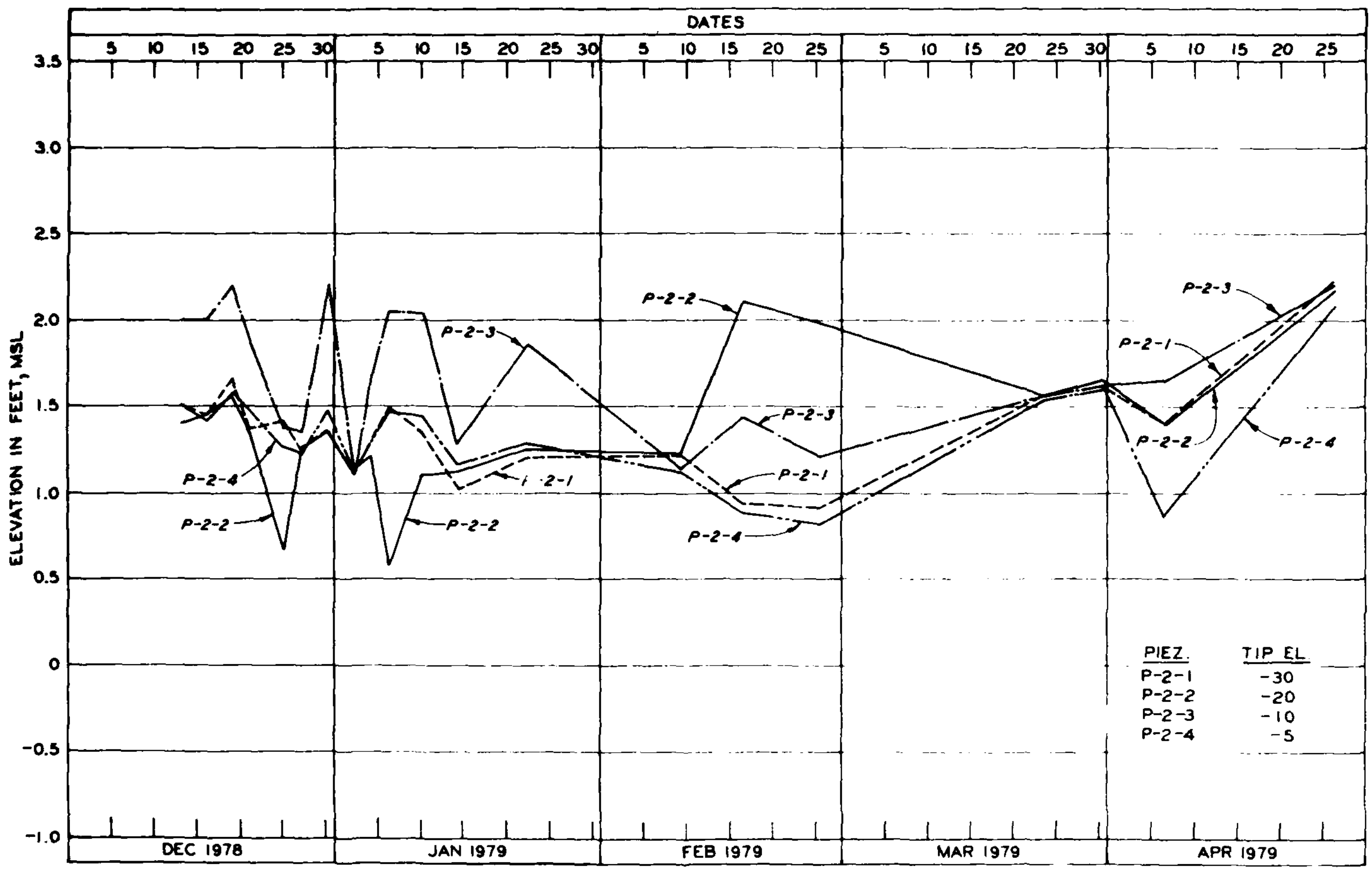



Figure 81. Pore Pressure versus Time for Piezometers P-2-1 through P-2-4

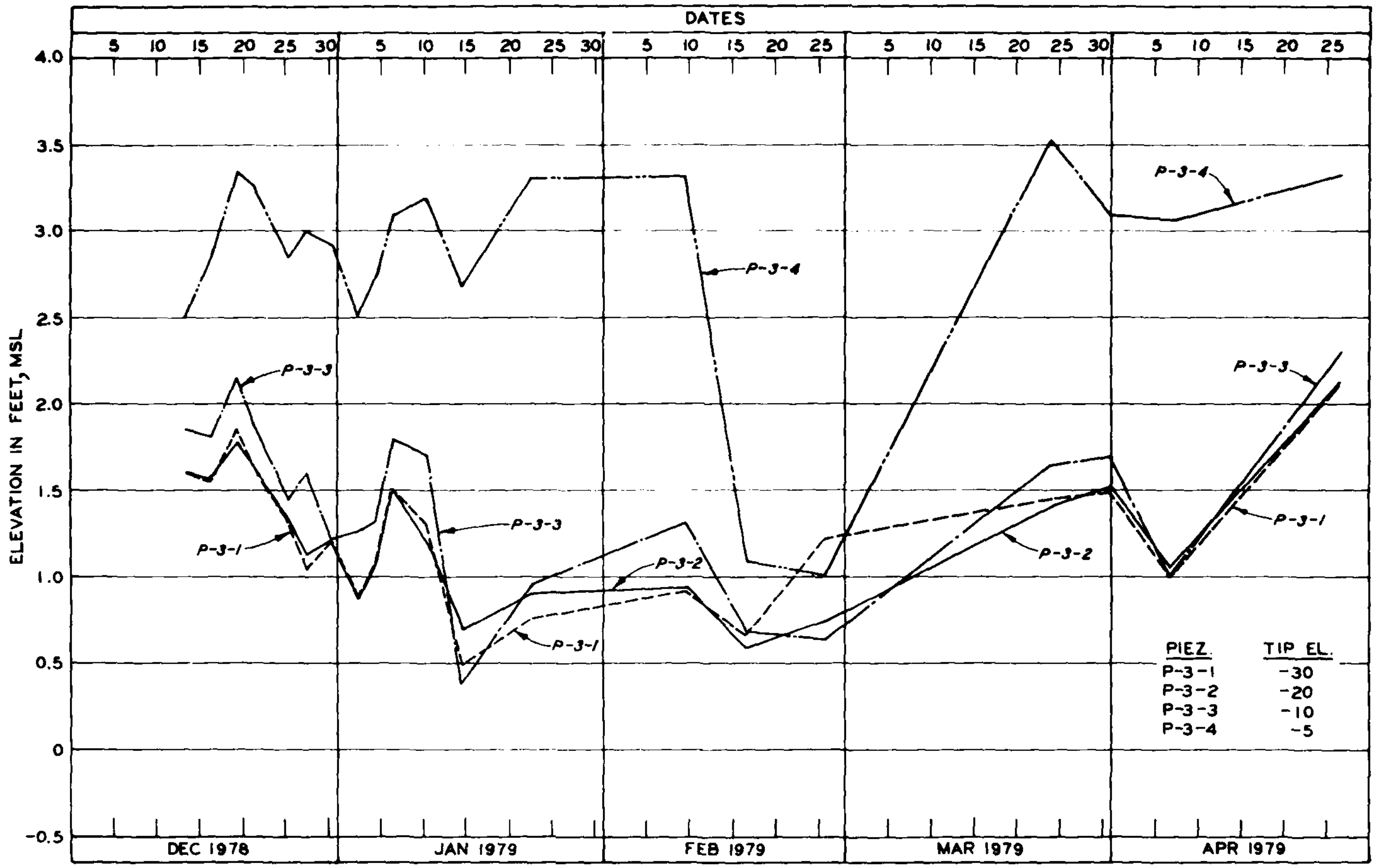



Figure 82. Pore Pressure versus Time for Piezometers P-3-1 through P-3-4

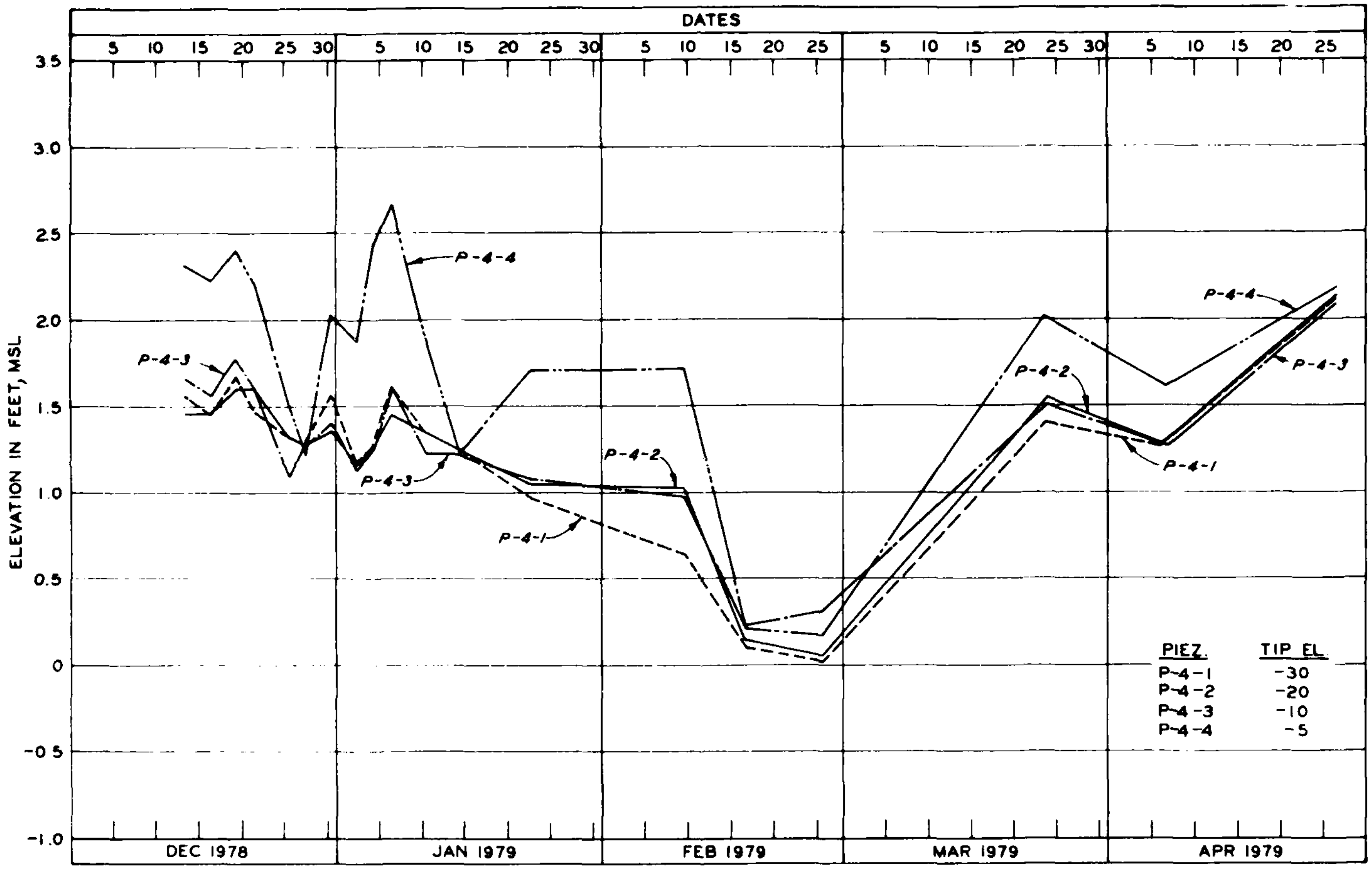



Figure 83. Pore Pressure versus Time for Piezometers P-4-1 through P-4-4

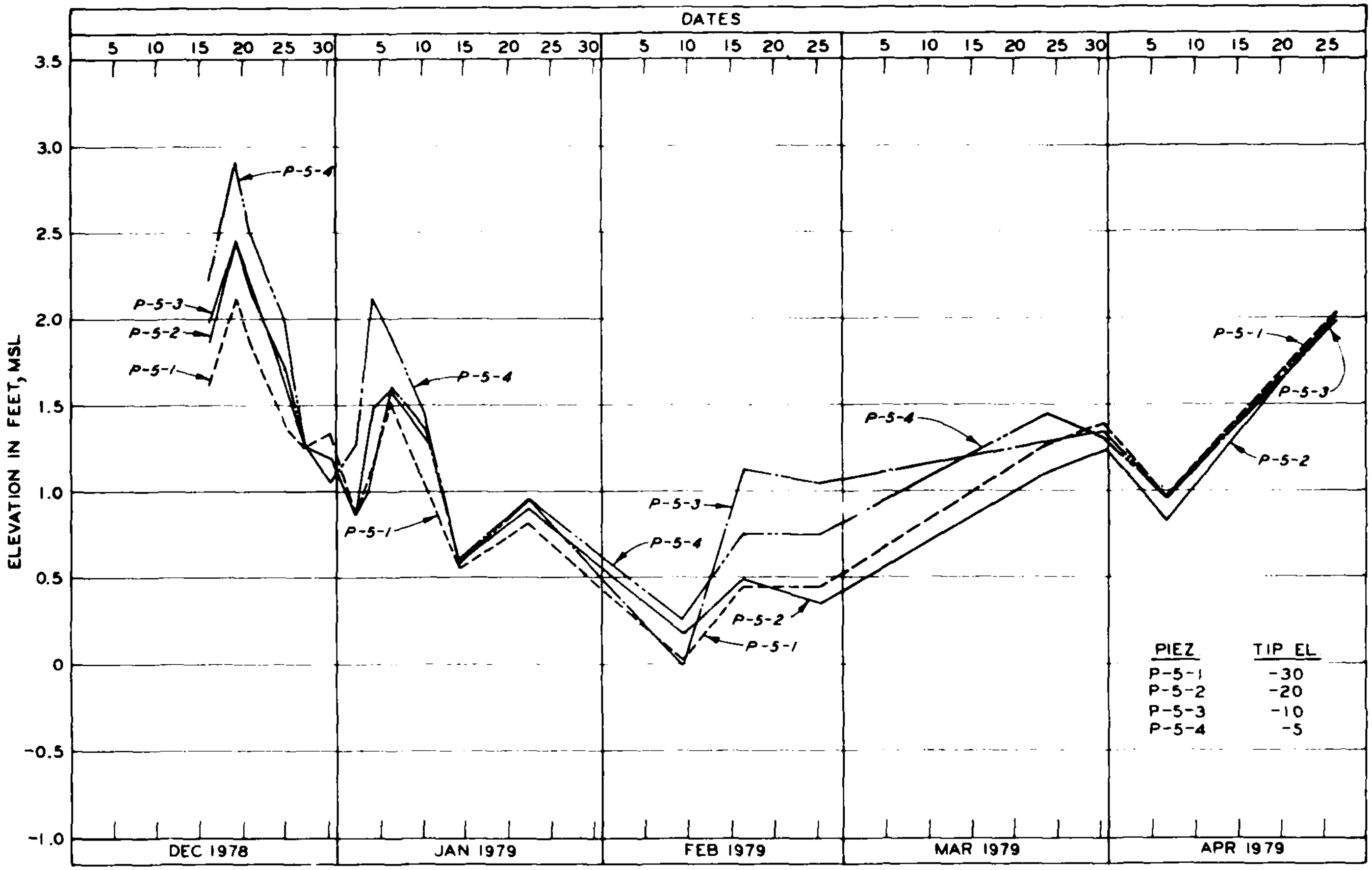



Figure 84. Pore Pressure versus Time for Piezometers P-5-1 through P-5-4

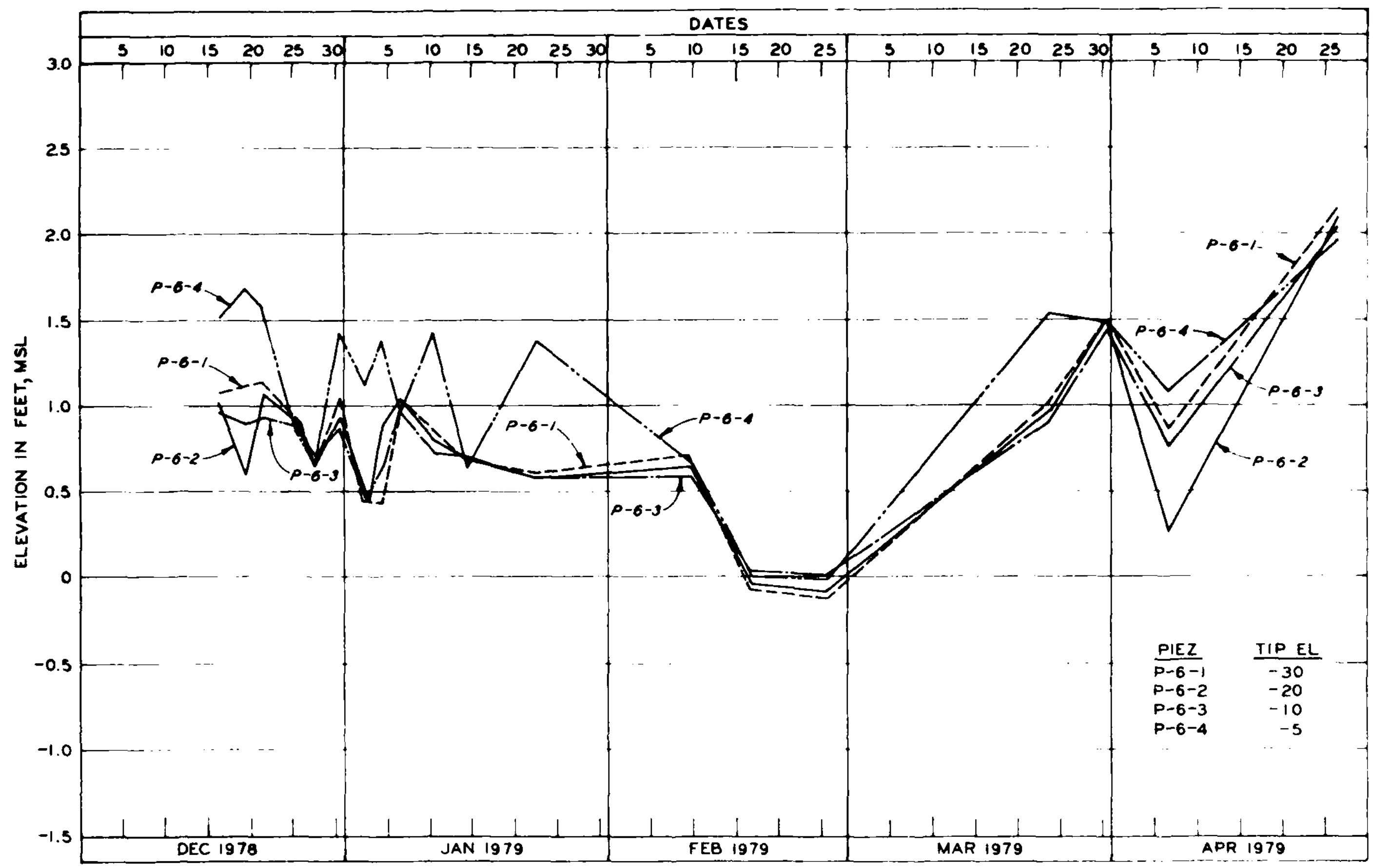



Figure 85. Pore Pressure versus Time for Piezometers P-6-1 through P-6-4

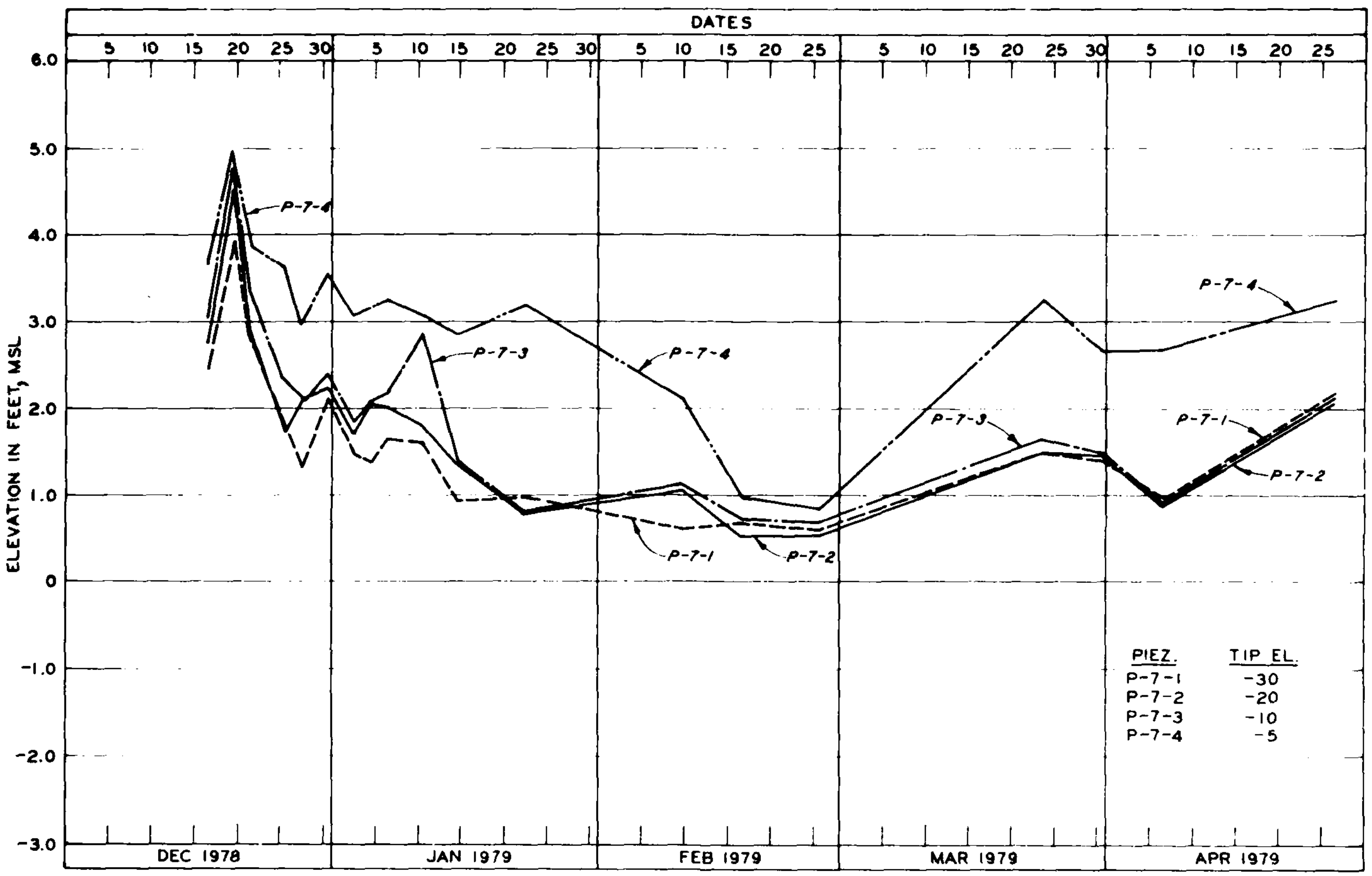



Figure 86. Pore Pressure versus Time for Piezometers P-7-1 through P-7-4

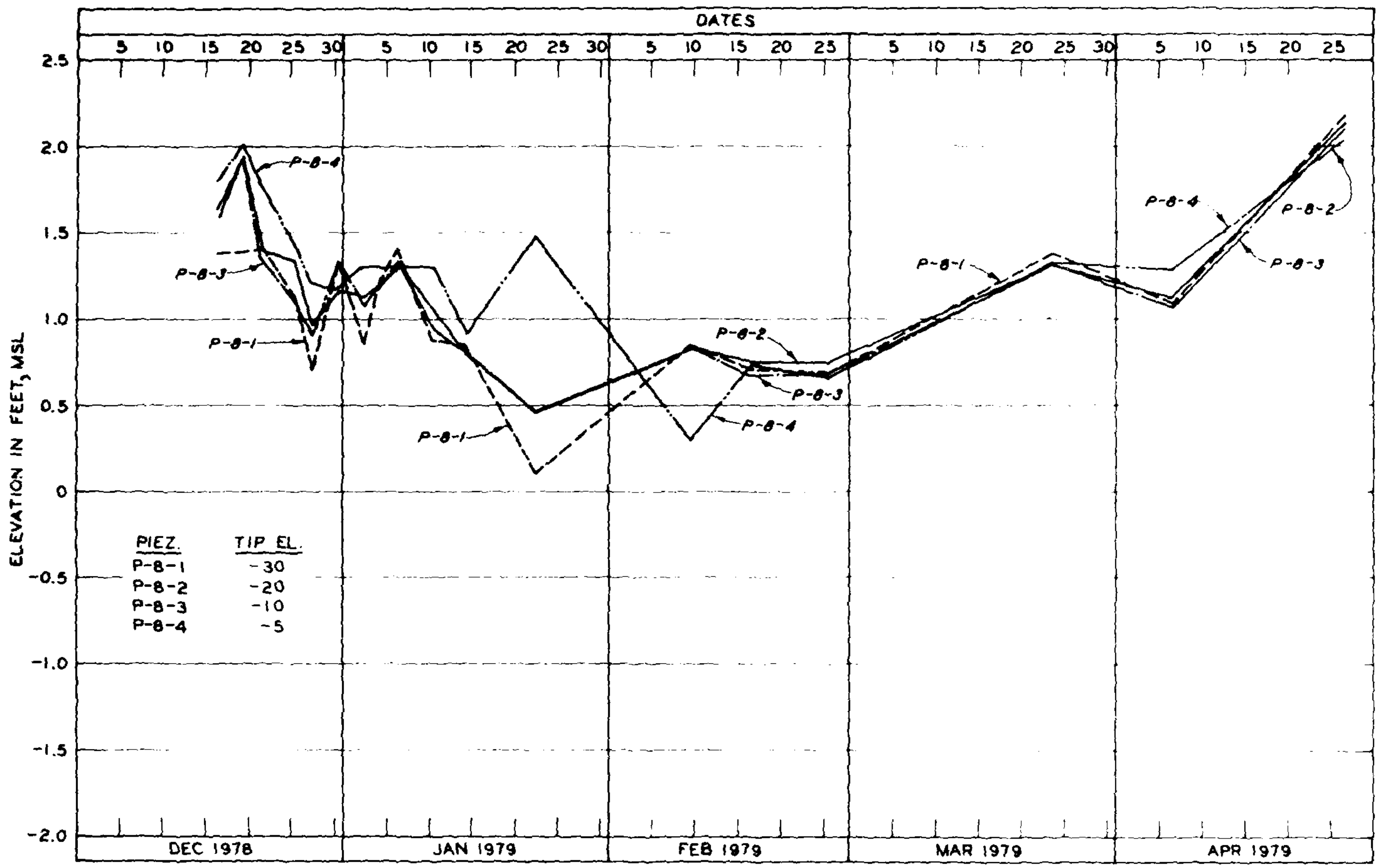



Figure 87. Pore Pressure versus Time for Piezometers P-8-1 through P-8-4

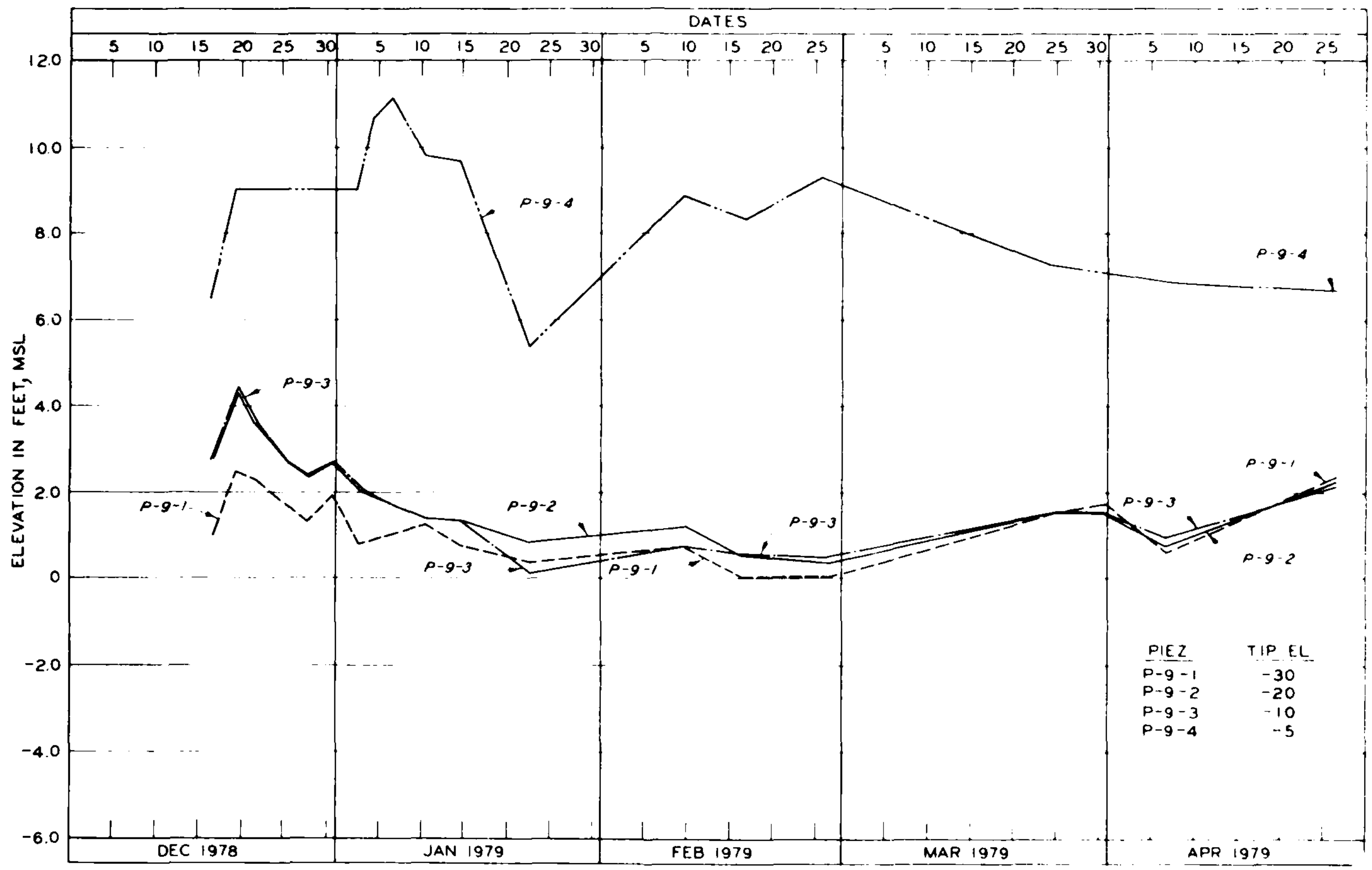



Figure 88. Pore Pressure versus Time for Piezometers P-9-1 through P-9-4

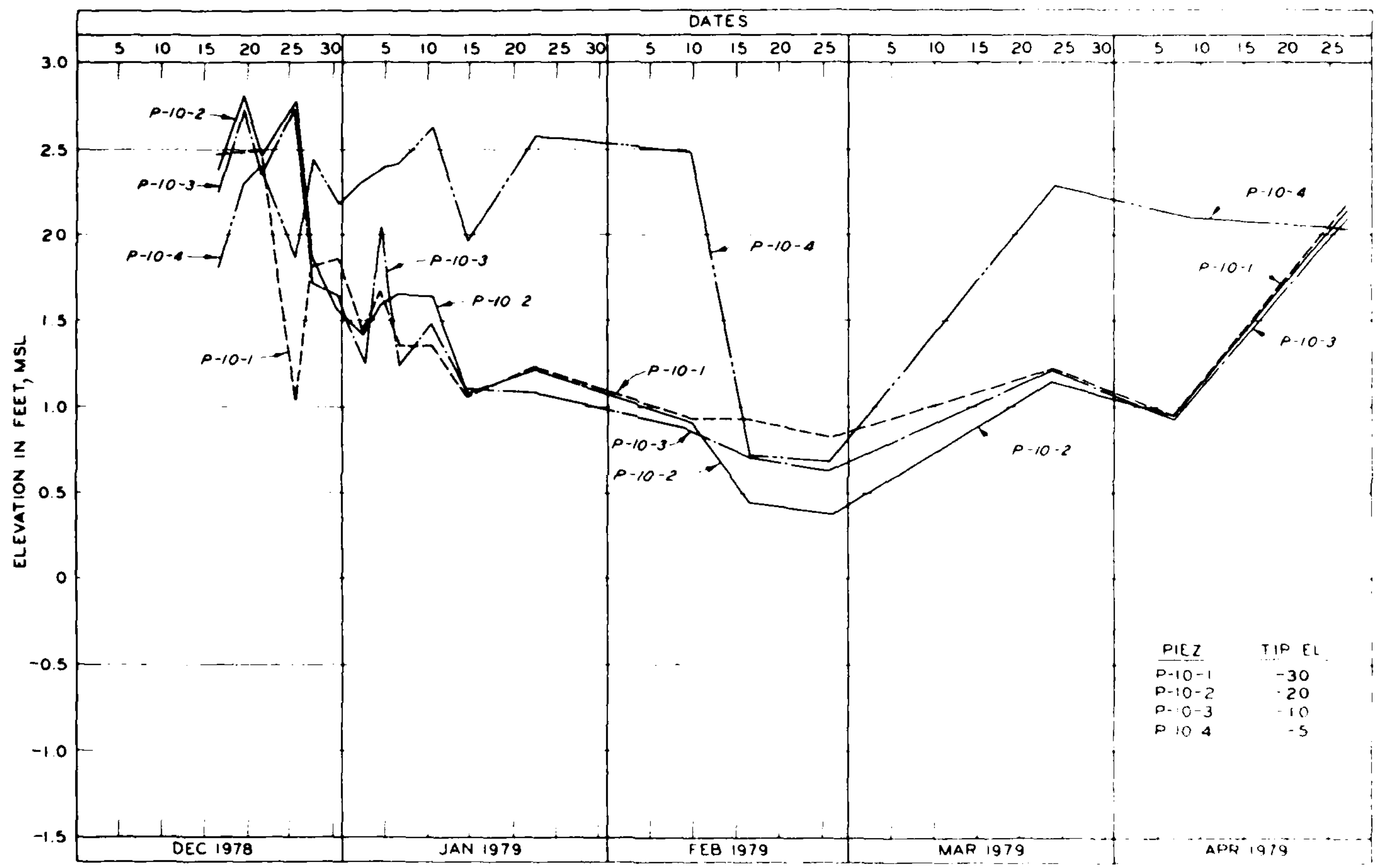



Figure 89. Pore Pressure versus Time for Piezometers P-10-1 through P-10-4

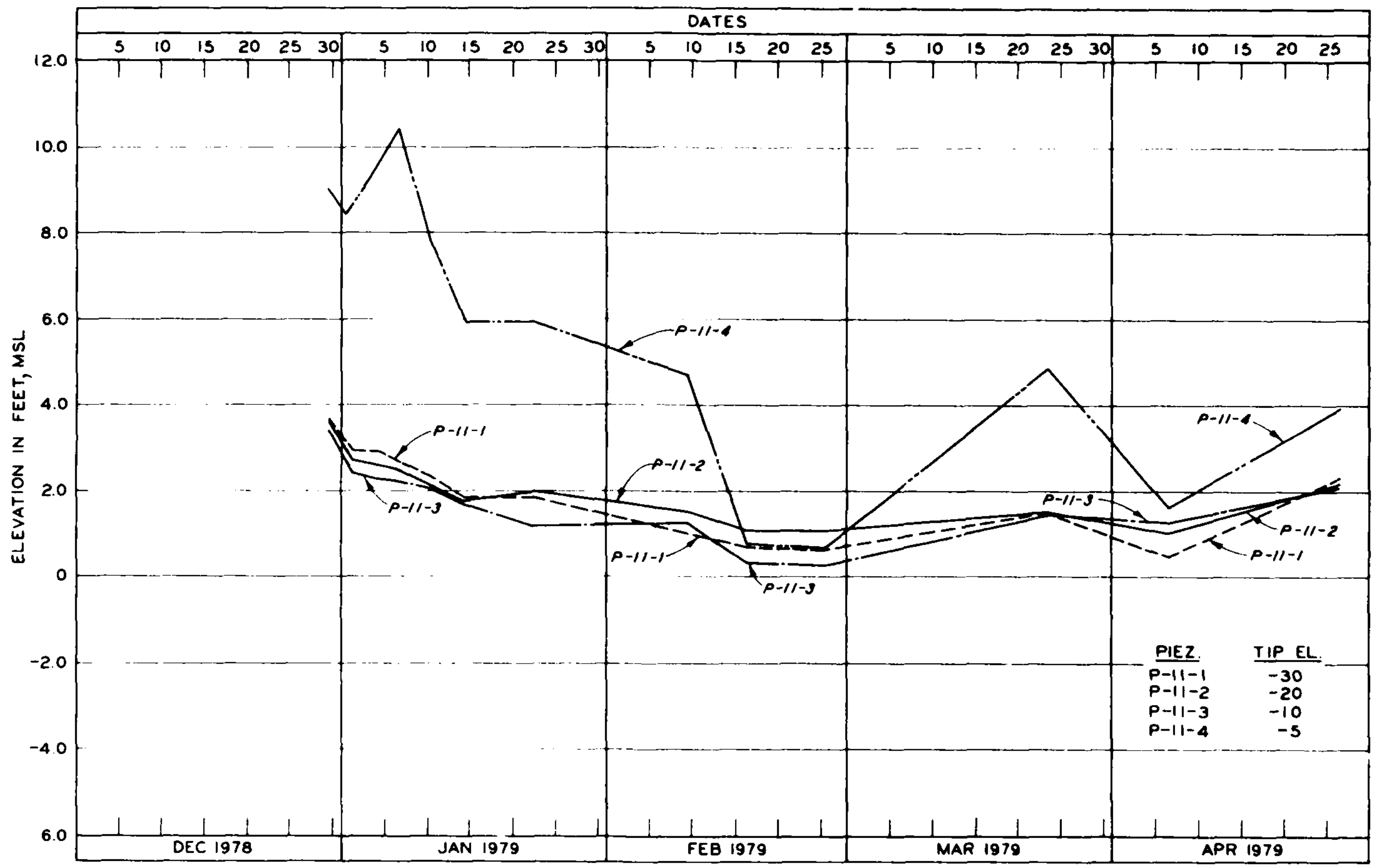



Figure 90. Pore Pressure versus Time for Piezometers P-11-1 through P-11-4

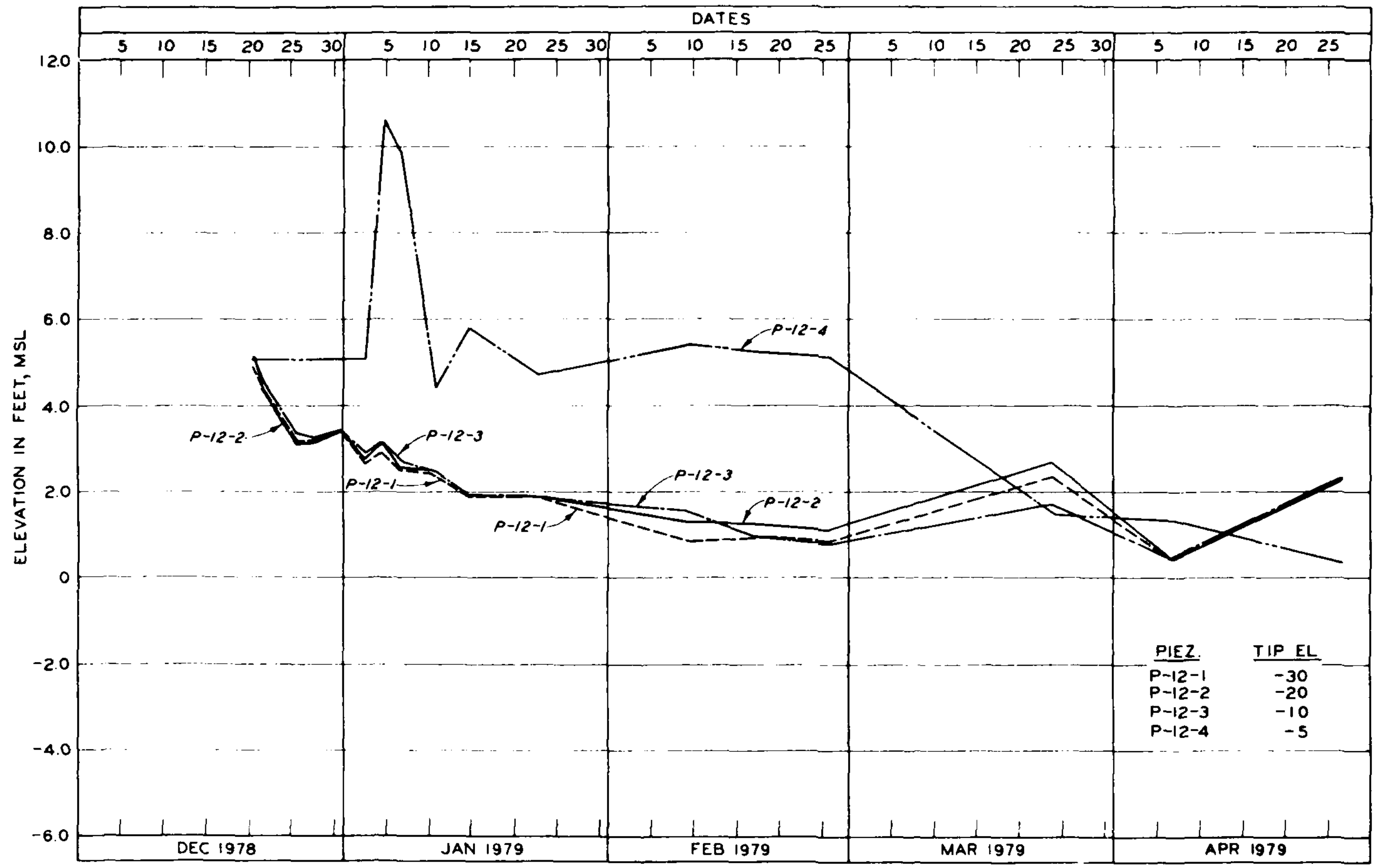



Figure 91. Pore Pressure versus Time for Piezometers P-12-1 through P-12-4

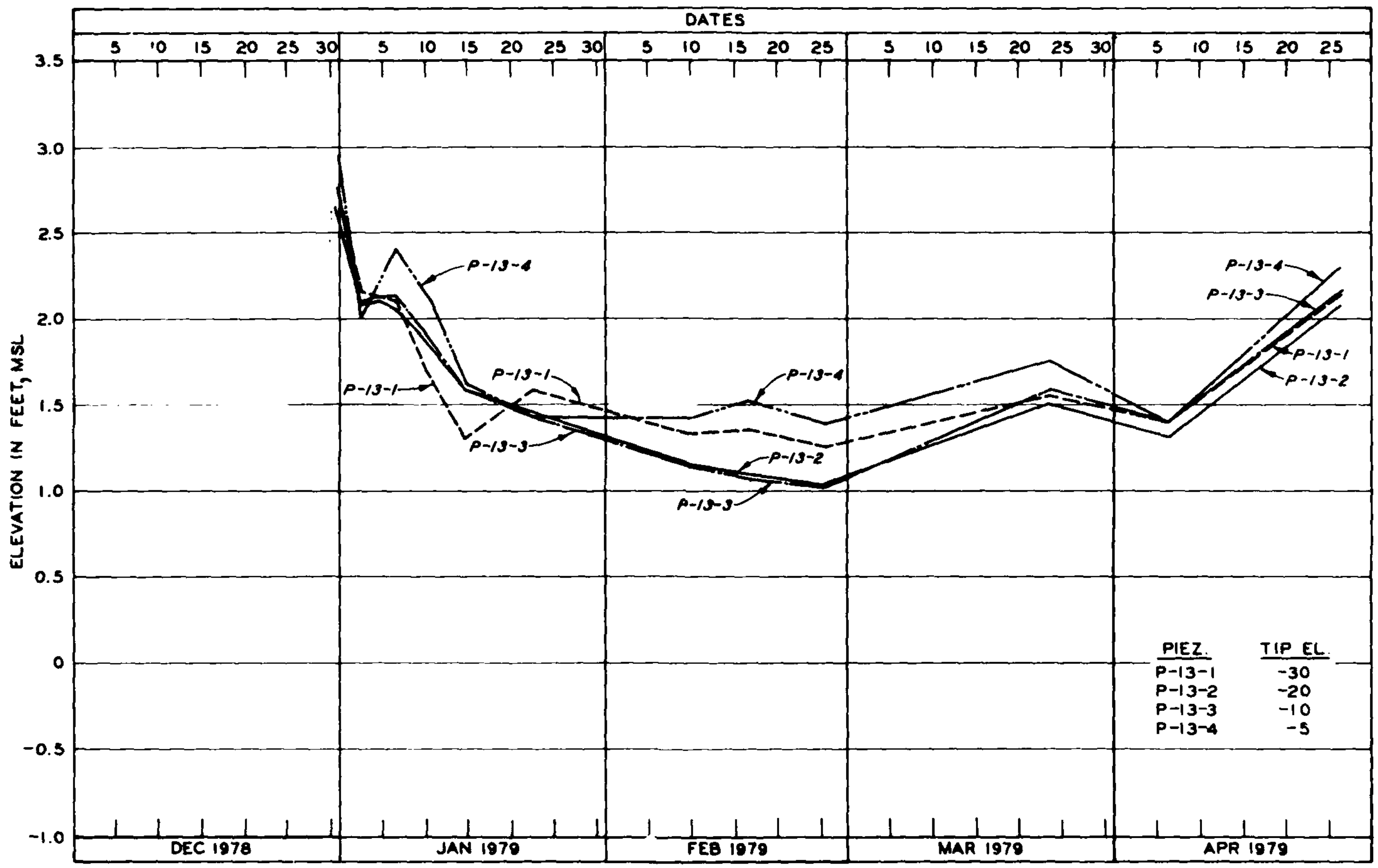



Figure 92. Pore Pressure versus Time for Piezometers P-13-1 through P-13-4

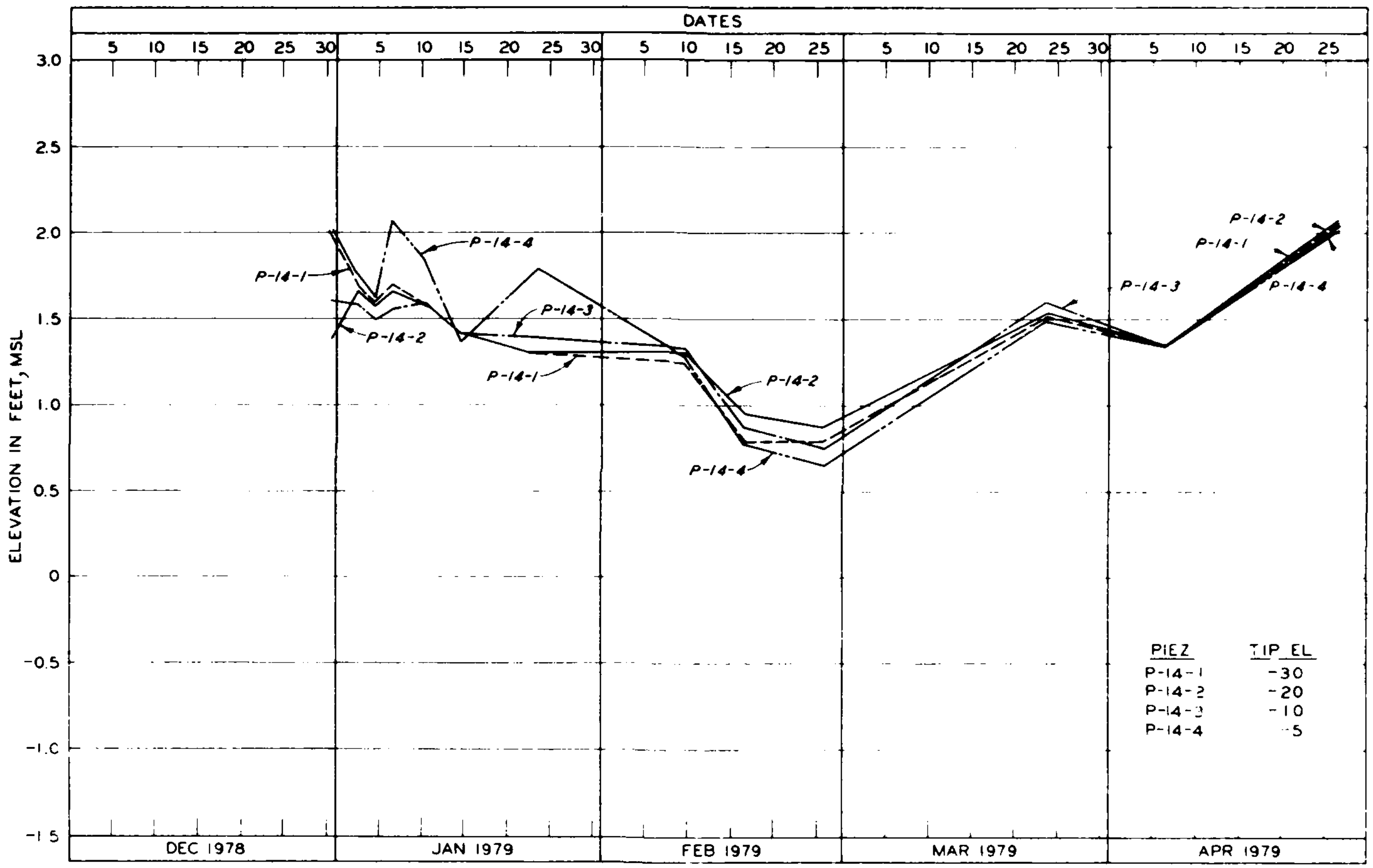



Figure 93. Pore Pressure versus Time for Piezometers P-14-1 through P-14-4

# APPENDIX D

# EMBANKMENT STABILITY ANALYSES

## Introduction

The purpose of this appendix is to present the results of the simplified Bishop analysis procedure conducted for the fabric-reinforcement embankment test section at Pinto Pass. Generally, this method assumes shear failure along an arc, although other failure shapes may be utilized. In this method the sliding mass is divided into slices of unit width, and a number of trial arcs are investigated to determine which is most critical. Details of this procedure are outlined in Engineer Manual EM 1110-2-1902.[1]

## Basic Assumptions for Analysis

For analysis of the embankment test section, the basic assumptions in the WES-developed computer program SAVA104 were as follows:

1. The simplified Bishop analysis method was assumed to be valid to determine the stability of a fabric-reinforced embankment.

2. The failure arc or plane was assumed to be tangent to the lower boundary of the soft clay that was assumed to be located immediately below the embankment resting on a layer or layers relatively stronger than the top layer.

3. It was assumed that the fabric strength would be equivalent to the strength of a cohesive clay layer uniformly distributed along the

failure arc or plane. The cohesive strength of the layer was assumed to be equivalent to the tensile strength of the fabric and the angle of internal friction was assumed to be zero. Details of the assumed behavior of the fabric at the interface of the fabric and fracture face are shown in Figure 94.

4. Critical failure arcs for an embankment with and without fabric reinforcement were assumed to be identical.

5. As-built dike geometries were assumed in the analyses and are shown in Figure 94. The crest width was fixed at 12 ft and the side slope width for all cases and stages of construction was assumed to be 70 ft. In this analysis the first stage or as-built was initially considered to be 7 ft above the surface of the reinforced fabric layer and the two subsequent layers to be constructed in the future were assumed to be added to the first. Therefore, the side slopes, cot $\beta$, are 10, 6.36, and 1.67, respectively, for dike heights of 7, 11, and 15 ft. The foundation layer thickness was also treated as a parameter in this study.

6. Foundation soil properties for the embankment materials were assumed to be constant, and it was assumed that no tension cracks occurred in the dike. Tension cracks due to settlement and lateral spreading were investigated, and it was determined that if cracking were assumed to occur, the driving or active force of the soil mass in a circular arc would be reduced. The soil properties of the soft foundation materials were considered to be parameters, and the cohesionless embankment material was considered to be constant with a cohesion strength $c = 0$ and an angle of internal friction $\phi = 30$ degrees.

7. Only the end-of-construction case was considered with the groundwater assumed to be at the same elevation as the fabric reinforcement layer.

## Treatment of Fabric Strength

Figure 94 shows that the fabric was laid flat on the soft underlying foundation materials beneath the base of the embankment and the potential failure plane extended through the toe of the embankment. Resistance was provided by the tensile strength of the fabric embedded beneath the embankment and was assumed to act uniformly along the length of the embedded arc length beneath the fabric. Therefore, the total resistance may be mathematically expressed as the sum of the resistance contributed by the fabric and cohesive resistance of the soil or:

$$c = c_f + c_u$$

where $c$ = total cohesive strength, ksf

$c_f$ = equivalent fabric cohesive strength, ksf

$c_u$ = soil cohesive strengths, ksf

The relationship between the fabric tensile strength $T_f$ and the equivalent fabric cohesion $c_f$ is determined by the following expression:

$$T_f = c_f L_f$$

where $L_f$ = length of the failure arc embedded in the foundation materials beneath the fabric reinforcement. Therefore, it can be seen that the total resistance, R, for each linear foot of the fabric and foundation soil is as follows:

$$R = L_f c$$

$$R = L_f \ (c_f + c_u)$$

$$R = L_f \ (\frac{T_f}{L_f} + c_u)$$

$$R = T_f + L_f c_u$$

## Parameter Investigation

To study the influence of the various parameters such as the height of the embankment H, the thickness of the soft foundation layer h, and the variables $c_f$ and $c_u$ defined earlier, it was necessary to develop design charts to illustrate their behavior and relationship to each other. Therefore, it was necessary to introduce dimensionless numbers by combining the above parameters as follows:

1. The depth ratio (D) is the sum of the embankment height (H) and foundation layer (h) divided by H or $D = \frac{(H+h)}{H}$. A reference line is drawn horizontal and tangent to the top of the embankment crest and dimensions are taken from this line. Figure 95 shows the depth ratio D versus various foundation thicknesses h for different embankment heights H.

2. In conventional slope stability problems, the stability number (N) is defined as $N = \frac{c}{\gamma H}$ where $\gamma$ is the density of soft foundation soil and c and H are as defined previously.

For a given set of parameters, a critical arc is established first, then the factors of safety and total cohesion are determined for a given arc. All computations for each set of conditions were conducted with the use of the WES computer, and typical results for one of several plots

necessary to develop the design curves for H = 7 are shown in Figure D3. Several curves were constructed for dike heights of 7, 11, and 15 ft and for various foundation layer thicknesses. For example, for a given foundation soil cohesion $c_u$, dike height of H = 7.0 ft, and foundation thickness h = 12 ft, the required fabric strength $T_f$ necessary to prevent embankment failure for a given safety factor may be determined from the left-hand side of Figure 96 (see example on figure). When the combined strengths of the soil cohesion and fabric layer are known, then the right-hand side of Figure 96 may be used to determine the safety factor of the embankment and subsequently the resistance $T_f$ needed to maintain embankment stability or equilibrium. Once the value of T is determined, then the number of sheets or layers of fabric can be found by simply dividing the allowable strength of the fabric into the value of T.

## Design Curves

Since the geometry of the test section was constrained by various design considerations, it was decided to include two sets of design charts: one set included the dimensions for the design problem at Pinto Pass and the other set for dimensionless application for H = 7.0 ft. The design curves shown in Figures 97 through 99 include the initial dike construction to el 8 (H = 7 ft) and two additional 4-ft incremental dike raisings to el 16 (H = 15 ft). These graphs were developed to show the relationship between the required fabric strength T and soil cohesion $c_u$ for safety factors of 1.0 and 1.3 for dike heights of 7.0, 11.0, and 15 ft. Therefore, if the designer specifies a safety factor between 1.0 and 1.3, Figures 97 through 99 may be used directly to

determine the required fabric strength T by entering the appropriate depth ratio D and unit cohesion $c_u$ for the foundation soil materials. For intermediate cases between a safety factor of 1.0 and 1.3, some interpolation may be necessary to determine the fabric strength.

Dimensionless design curves 1 and 2, prepared from several computer runs for a dike height of H = 7.0 ft, are shown in Figures 100 and 101 to determine the proper fabric strength T necessary to provide embankment equilibrium and to prevent failure. A flowchart and description of the basic definition for the components of the soil and fabric stability numbers N are shown in Figure 102. A sample after-construction problem for embankment height, H = 8.0 ft, for the Pinto Pass embankment is as follows:

Given three geometrical parameters (refer to dike drawing on Figure 100):

(1) Embankment slope 1:10 or Cot $\beta$ = 10

(2) Embankment height H = 8.0 ft

(3) Soft foundation layer h = 12.0 ft

and three soil parameters:

(1) Density of embankment materials $\gamma_H$ = 100 pcf

(2) Density of soft foundation materials $\gamma_h$ = 90 pcf

(3) Cohesive strength of soft foundation soil $c_u$ = 0.05 ksf

Specified safety factor FS = 1.3

Required: fabric tensile strength T

Solution: Find depth ratio D

$$D = \frac{H+h}{H} = \frac{8.0 + 12.0}{8} = 2.5$$

From Chart 1, Figure 100, for a given safety factor FS = 1.3, the total stability number N equals 0.138

The component number $N_u$ for the soil cohesion is:

$$N_u = \frac{c_u}{\gamma_h H} = \frac{0.05\ \text{ksf}}{(0.09\ \text{kcf})(8\ \text{ft})}$$

$$N_u = 0.069$$

Therefore, the component number $N_f$ for the fabric is:

$$N = N_u + N_f$$

$$0.138 = 0.069 + N_f$$

$$N_f = 0.069$$

Then, the unit cohesion $c_f$ of the fabric is:

$$c_f = N_f \gamma_h H$$

$$c_f = (0.072)(0.09\ \text{ksf})(7.0\ \text{ft.})$$

$$c_f = 0.050$$

From Chart 2, Figure 101, the required fabric tensile strength T = 3.7 kip/ft-width or T = 310 lb/in.-width

## Composite Design Chart

A third and concluding design chart, Figure 103, was constructed to include all of the foregoing assumptions and design parameters necessary to determine the required fabric strength for a sand embankment located on a soft clay foundation of varying thickness. Figure 103 is a composite design chart for determining the fabric strength for incremental dike raising to heights of H = 7.0, 11, and 15 ft above the fabric, at a specified safety factor of 1.3. An example problem shown on this

design chart indicates that there are basically three geometrical design parameters required and three soil parameters necessary to determine the required fabric strength T for any soft foundation layer thickness h. This chart includes the more common range of parametric values encountered in soft ground engineering design problems for dredged material containment dikes constructed on most of the riverine and estuarine clay deposits found in many of the coastal districts.

ENDNOTE

[1]Department of the Army, Corps of Engineers. Engineering and Design, Stability of Earth and Rockfill Dams. Engineer Manual EM 1110-2-1902. Washington: Office of the Chief of Engineers (1970).

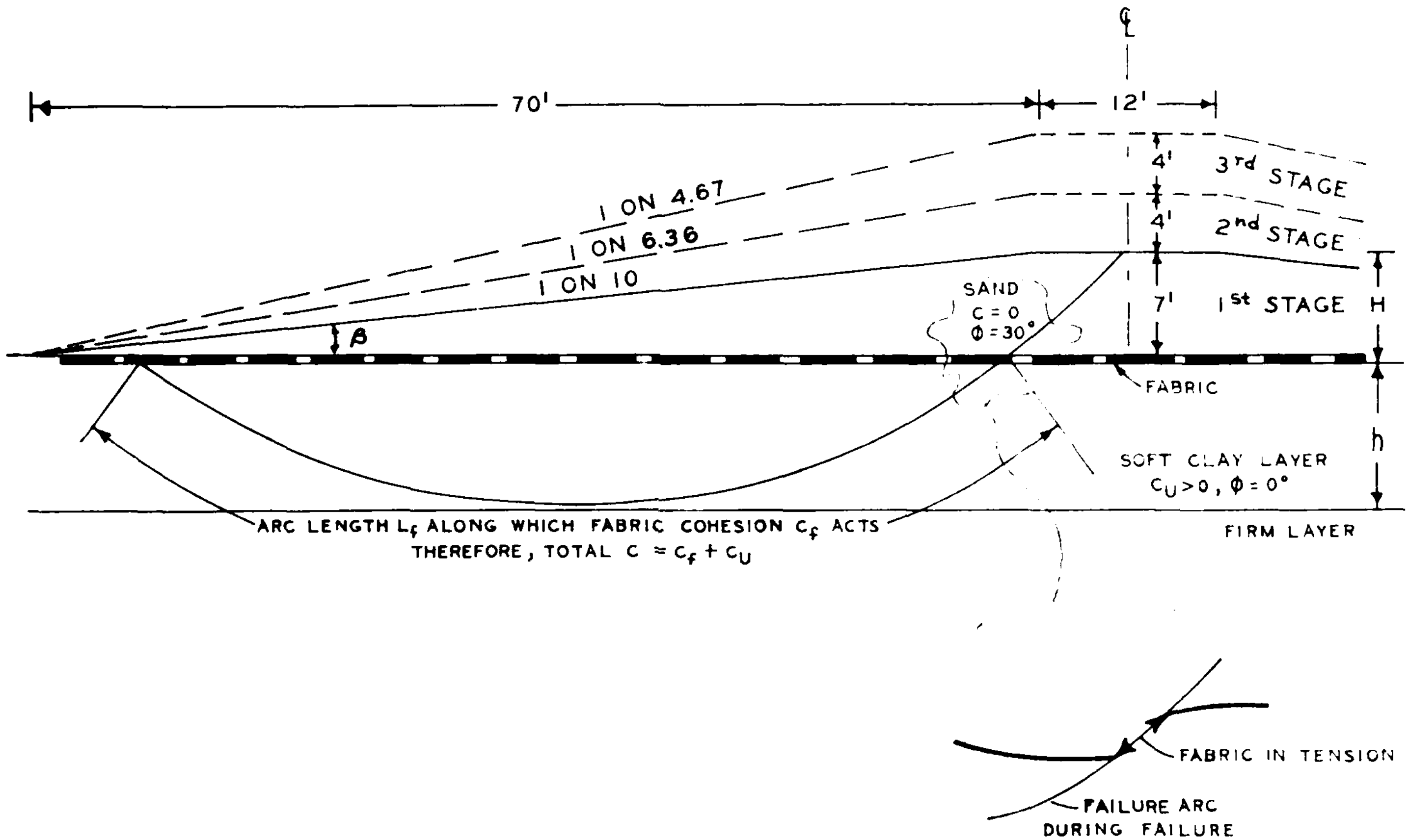



Figure 94. Fabric-Reinforced Embankment Test Section

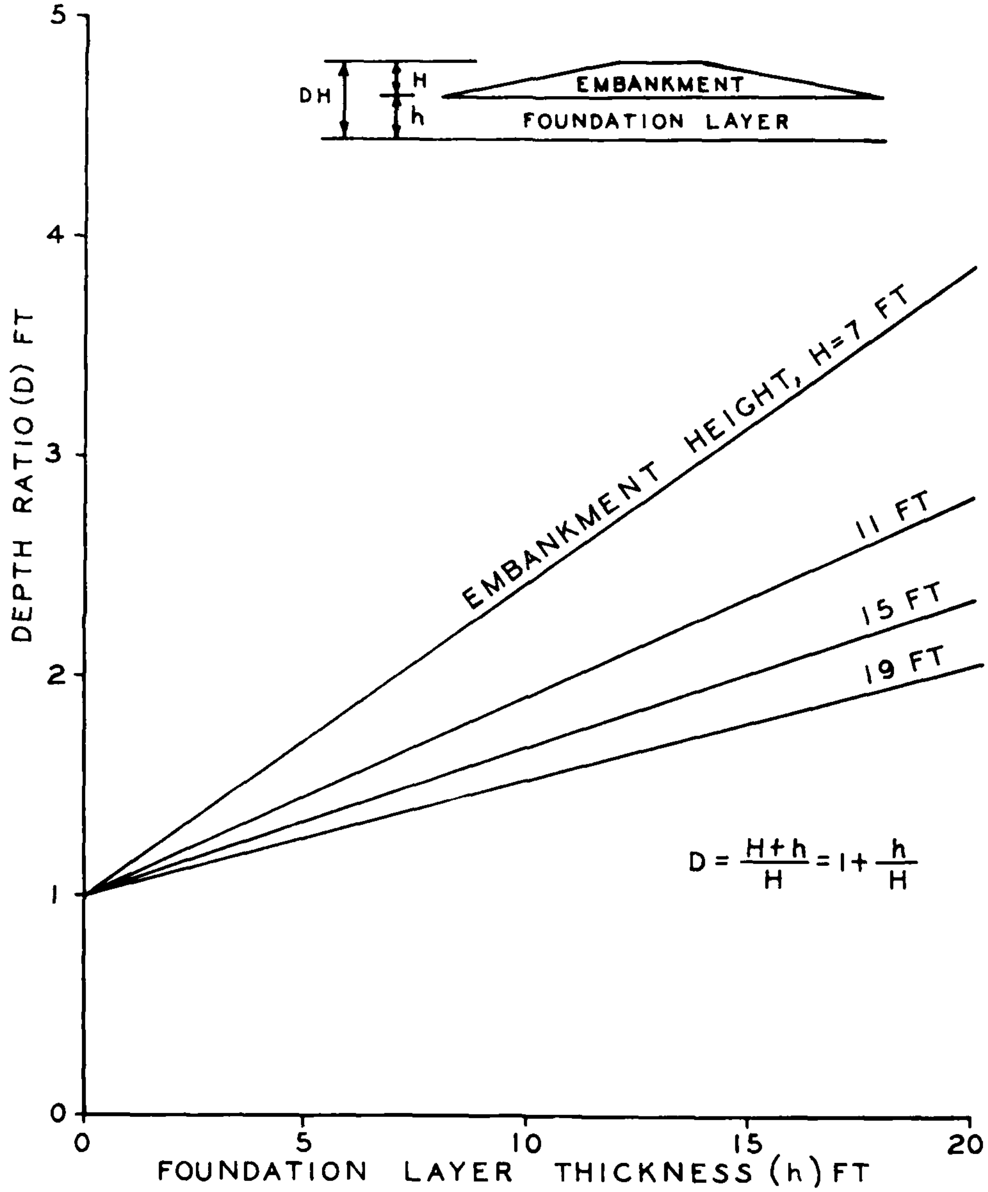



Figure 95. Depth Ratio D Versus Foundation Layer Thickness h

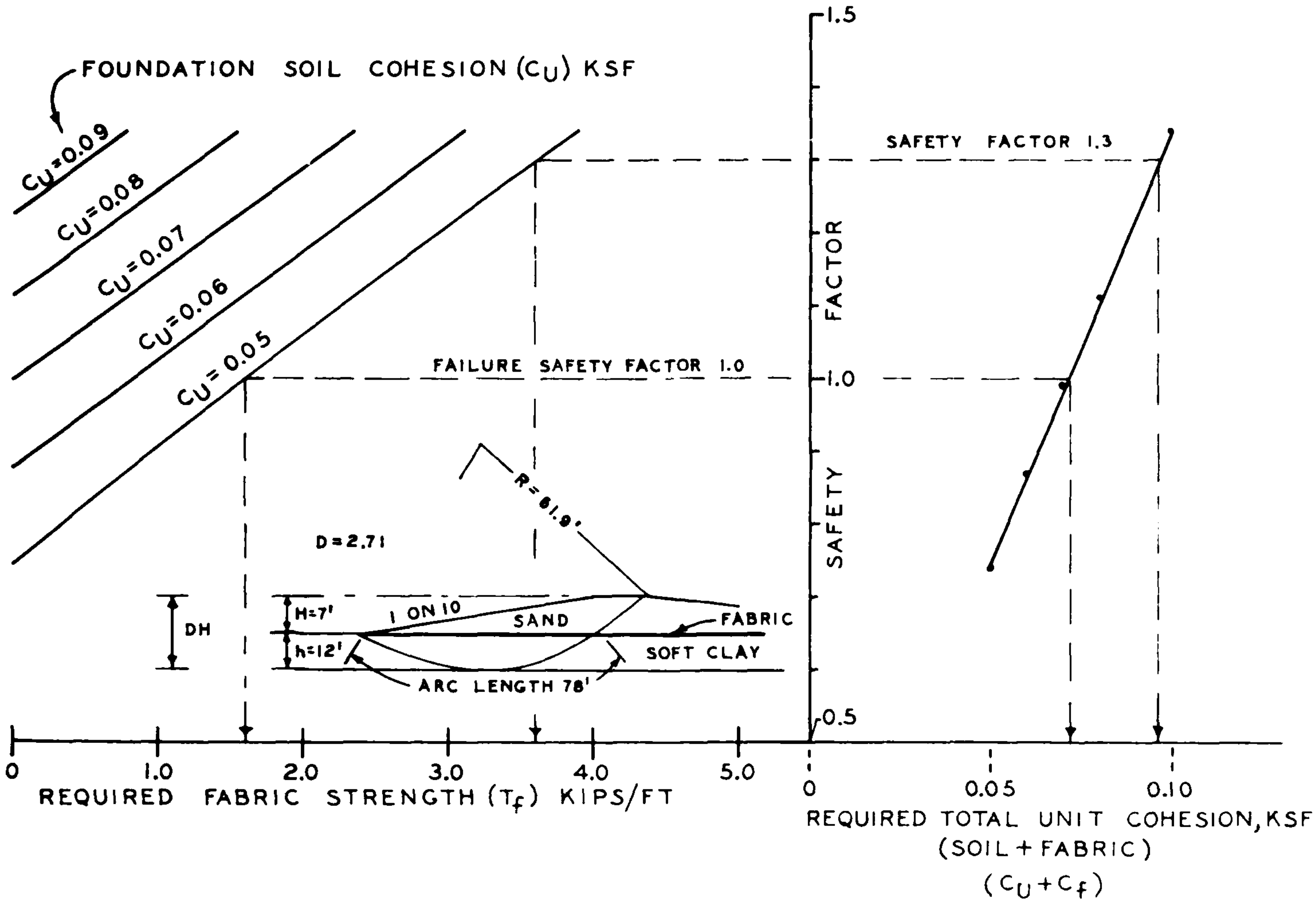



Figure 96. Typical Curves Used to Develop Design Curves

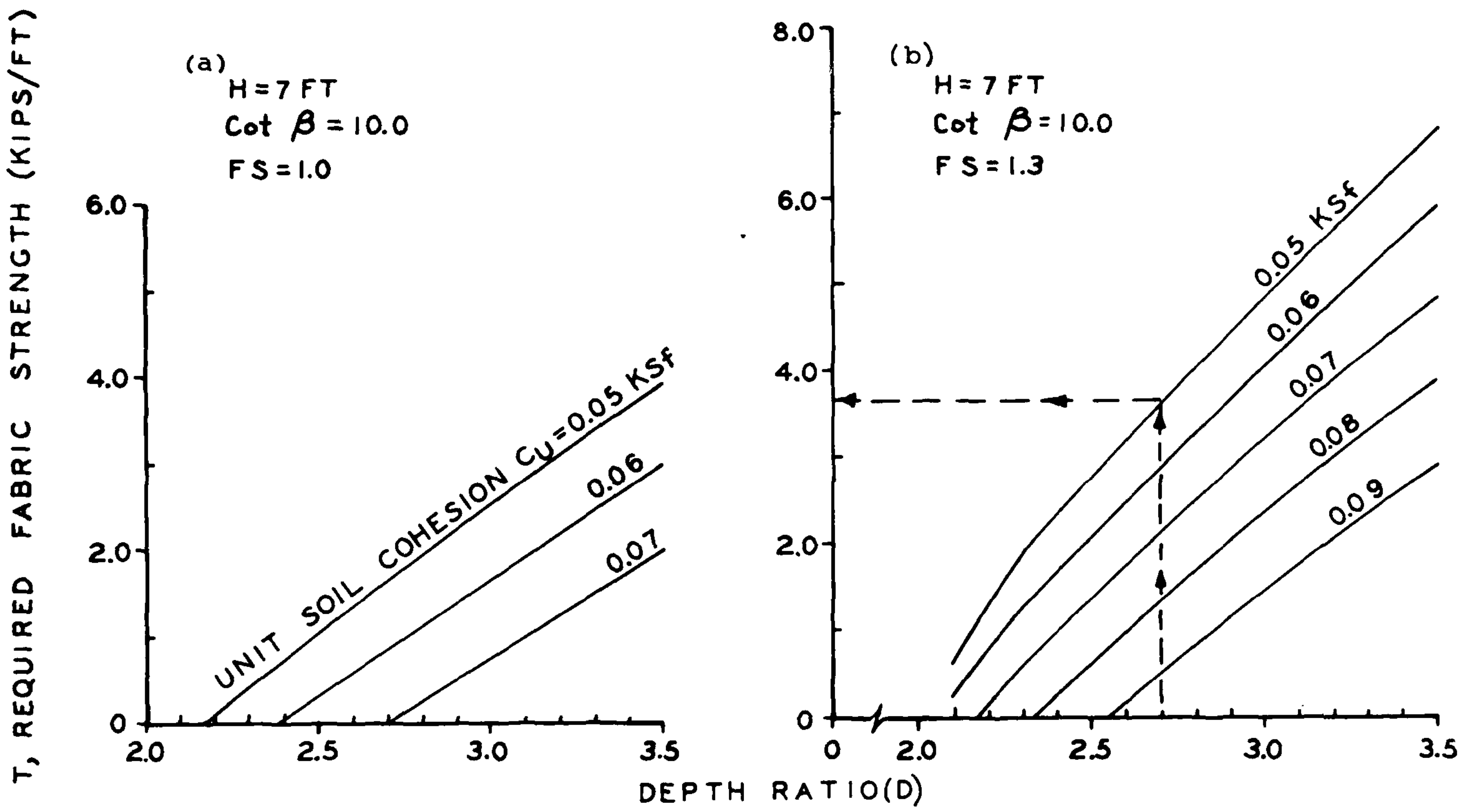



Figure 97. Design Curves for Dike Height = 7 ft

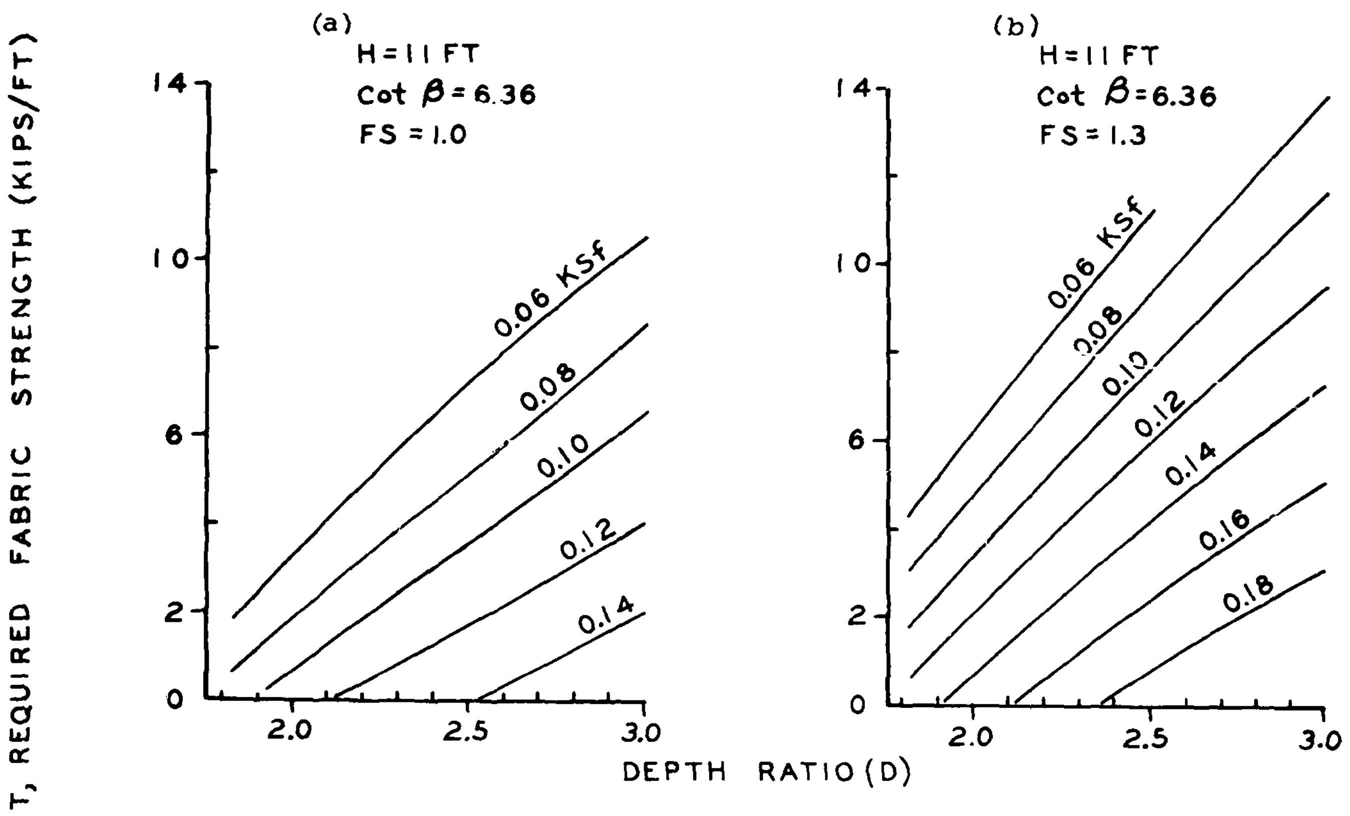



Figure 98. Design Curves for Dike Height = 11 ft

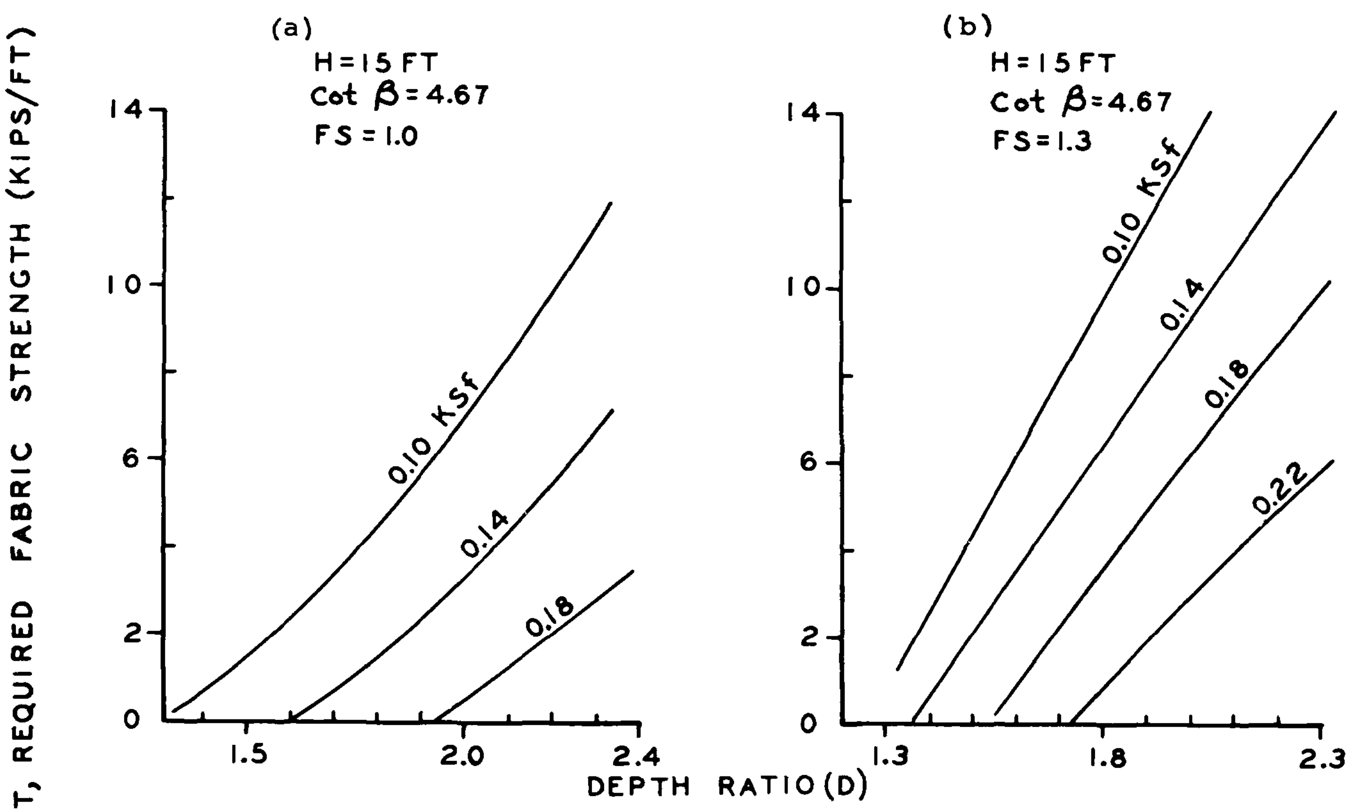



Figure 99. Design Curves for Dike Height = 15 ft

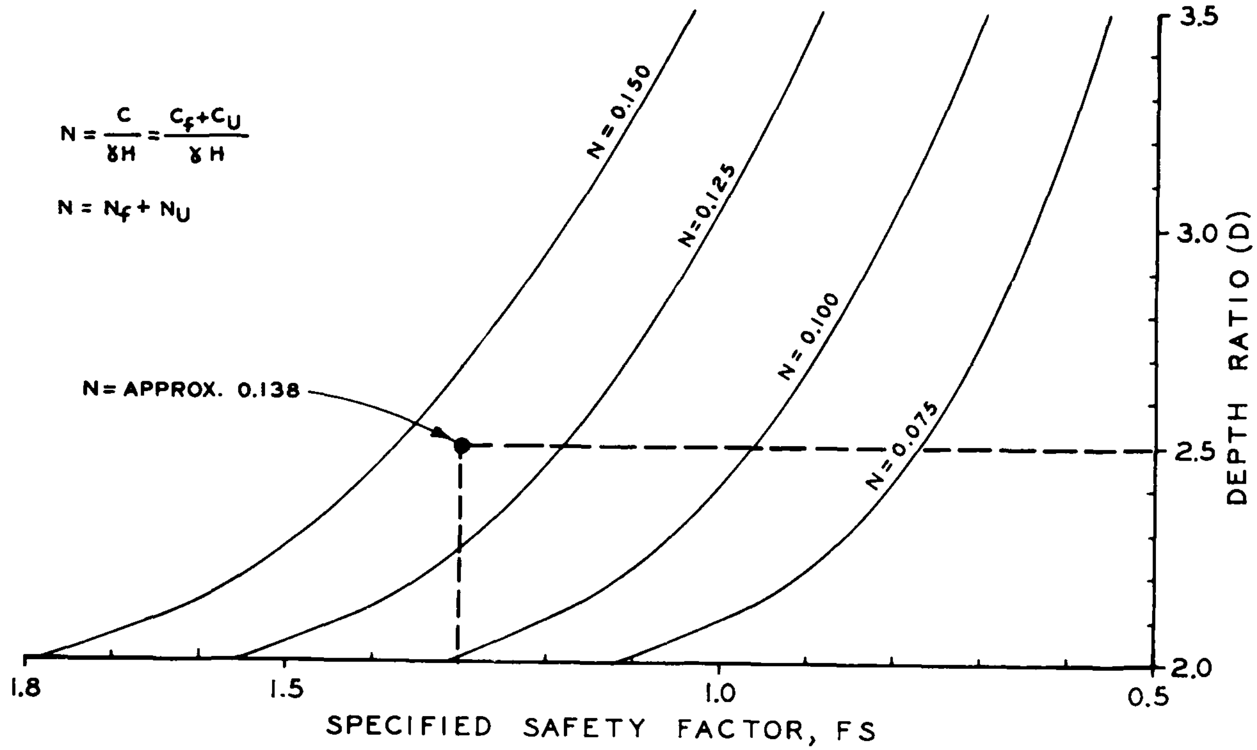



Figure 100. Design Chart 1 for Determining Stability Number N

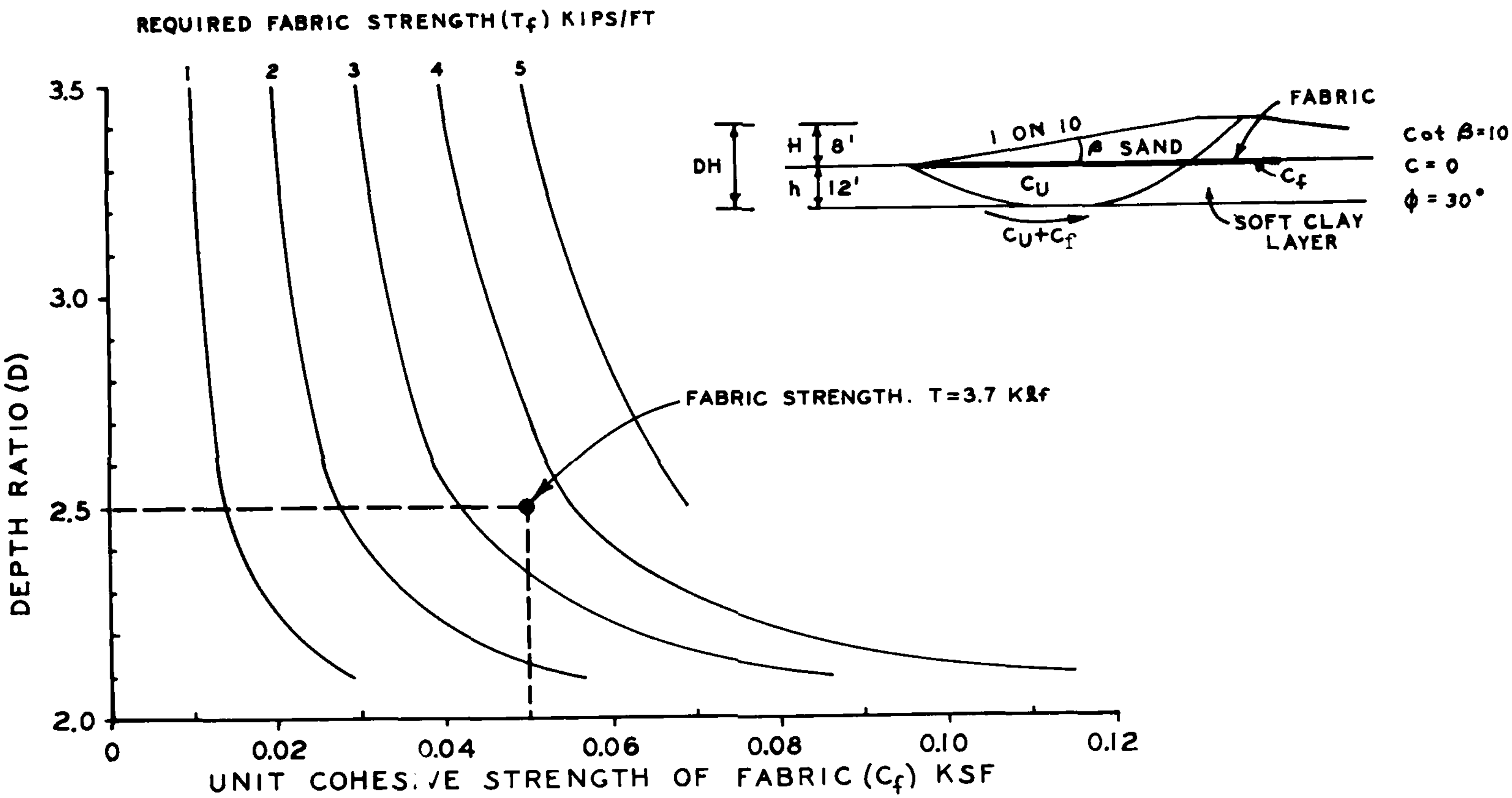
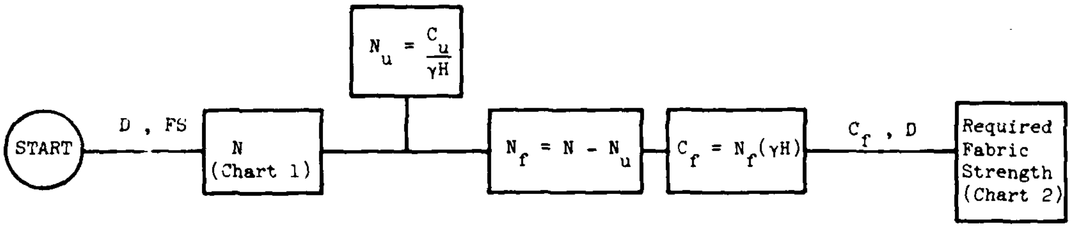



Figure 101. Design Chart 2 for Determining Fabric Strength $T_f$




$$N = \frac{(C_u + C)}{\gamma H} = \frac{C_u}{\gamma H} + \frac{C_f}{\gamma H} = N_u + N_f$$

$$N_f = N - N_u$$

$$C_f = N_f\ (\gamma H)$$

where

$N$ = stability number for combined contribution from soil and fabric

$N_u$ = component number for soil cohesion $C_u$

$N_f$ = component number for fabric cohesion $C_f$

$C_u$ = unit cohesion of soil, ksf

$C_f$ = unit cohesion of fabric, ksf

**Figure 102. Flowchart and Description of Basic Definitions for Dimensionless Fabric Design Charts 1 and 2**

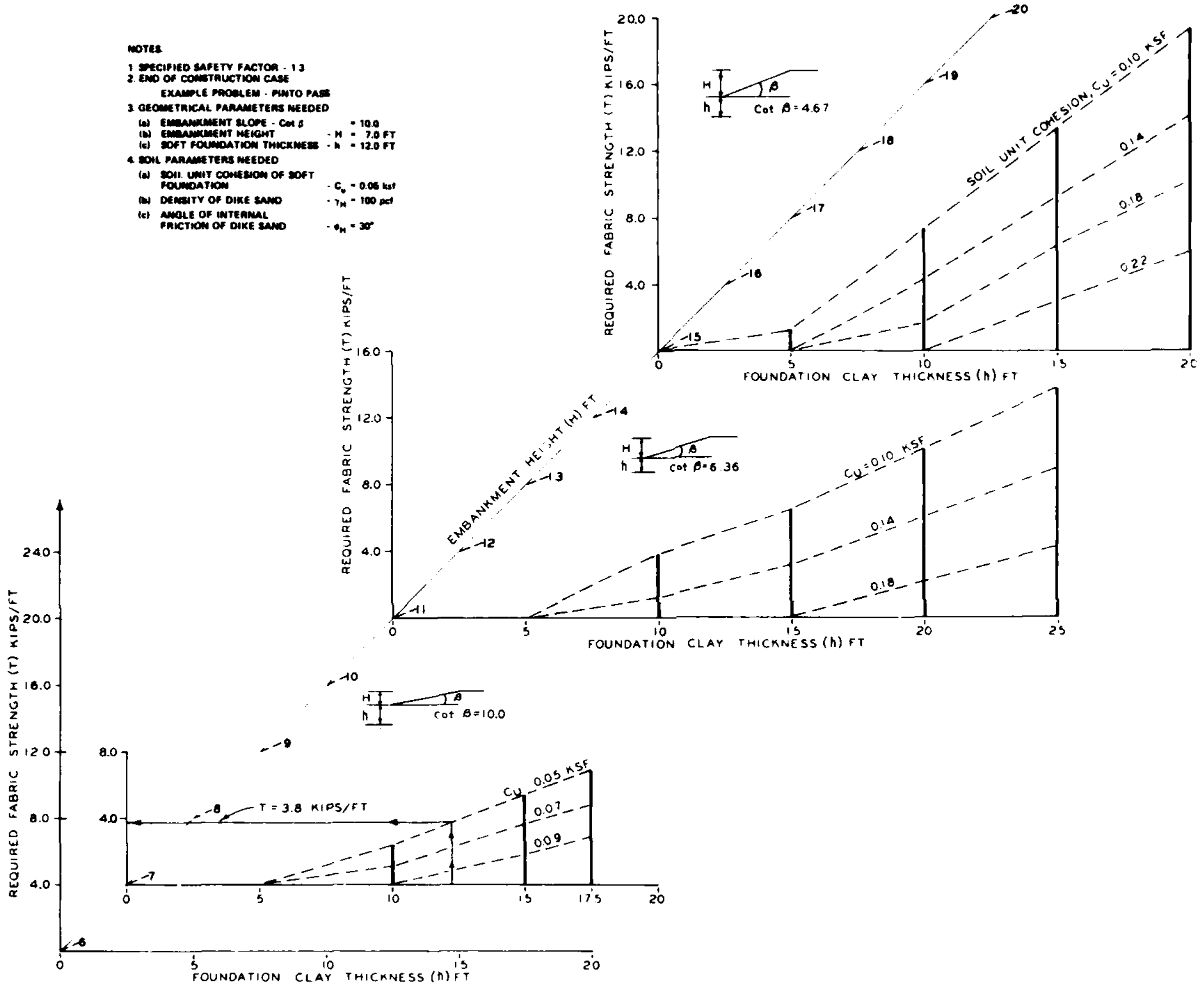



Figure 103. Design Curves to Determine Embankment Fabric Strength

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xiii, 225 p. : ill. ; 27 cm. -- (Technical report / U.S. Army Engineer Waterways Experiment Station ; EL-81-7)
Cover title.
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"Monitored by Environmental Laboratory, U.S. Army Engineer Waterways Experiment Station."

1. Dredging spoil. 2. Embankments. 3. Fills (Earthwork). 4. Mobile Bay (Ala.) I. United States. Army. Corps of Engineers. Mobile District. II. United States. Army.



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Design, construction, and analysis of fabric : ... 1981.
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Corps of Engineers. Office of the Chief of Engineers. III. U.S. Army Engineer Waterways Experiment Station. Geotechnical Laboratory. IV. Title V. Series: Technical report (U.S. Army Engineer Waterways Experiment Station) ; EL-81-7.
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